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Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

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Papers

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OUTLET STRUCTURES FOR FIXED-DISPERSION CONE VALVES[a]

Maurice L. Dickinson,[1] M. ASCE, Stanley M. Barnes,[2] A.M. ASCE
and Robert S. Milmoe, Jr.[3]
(Proc. Paper 1725)

SYNOPSIS

Following the computation of hydraulic capacity, selection of the most table valve and outlet structure for high-pressure regulation of large dis- ırges requires study and evaluation of site conditions and operating re- rements. This paper describes three fixed-dispersion cone valve installa- ıs which were designed to suit widely varying conditions and requirements :hree different locations; Santa Felicia Dam, Vermilion Dam, and Portal verhouse, all in California. The site conditions and operating require- nts at Vermilion were more severe than at the other two locations, and the rmilion installation therefore required more detailed design investigations l a more elaborate outlet structure. Hydraulic model tests were made for ;ign purposes and tests of the prototype installation were performed during :ial operation, thus providing a considerable amount of technical informa- ı. For these reasons, primary attention in this paper is devoted to the rmilion installation. The installations at all three locations, however, are cribed to illustrate satisfactory use of fixed-dispersion cone valves for a le range of site and operating conditions.

Valve Characteristics

The fixed-dispersion cone valve is one of the simplest types of high- ssure, free-discharge regulating valves. Figure 1 is a cutaway view of

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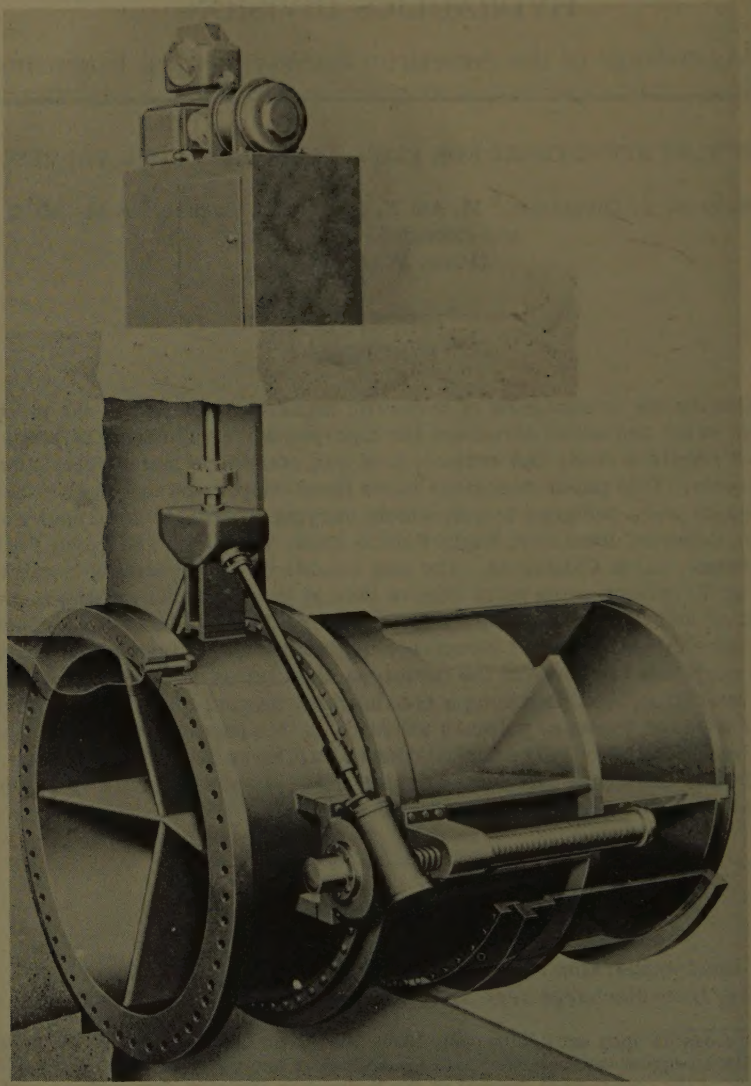


CUTAWAY VIEW OF HOWELL-BUNGER VAL

COURTESY OF S. MORGAN SMITH COMPANY

FIGURE

e valve, showing the principal features of its construction.

A clear description of the valve and its operation is presented by Elder nd Dougherty in their paper, "Characteristics of Fixed-Dispersion Cone alves," published as ASCE Transactions Paper 2567, Vol. 118, 1953. The llowing description is quoted almost verbatim from that reference: Essentially the valve . . . is a cylindrical gate mounted with the axis horiontal (for horizontal discharge). When open, the flow is deflected by a conial end piece mounted with the apex upstream and issues from the valve as a iverging hollow conical jet. . . . . The deflector cone is connected the valve body by vanes that connect the two parts and act as guides to the xternal sleeve. The external sleeve, which controls the valve opening, seats gainst the cone (when closed) and retracts over the outside of the valve body. he sleeve is moved by means of screws, connected by shafting and gearing the operating motor and controls. The position of this sleeve is shown on n indicator dial which is directly connected to the driving mechanism."

Some of the valve characteristics are inherently favorable for its use as a igh-pressure regulating valve for free-discharge. The most important of ese are:

1. The valve is simple in design and construction, and is correspondingly economical.
2. It is efficient hydraulically, having a discharge coefficient of about 0.85 at full gate opening, based on the formula $C_D = Q/A \sqrt{2 g H}$
3. The principal hydraulic forces on the sliding sleeve are radial and exert no significant motive force either to open or close the valve. There is accordingly no tendency for the valve to slam shut or open.
4. The operating mechanism is entirely external, which facilitates inspection and maintenance.
5. The flow is discharged in a wide-angle diverging hollow cone, thus dispersing the energy over a wide area and minimizing the problem of energy dissipation.

While these merits are substantial, the valve also has some disadvantages. he simple conical dispersion of the discharge, which provides the basis for ost of the favorable characteristics, is also responsible for the principal ndesirable characteristics:

1. On installations where the discharge is not confined, the diverging jet will spread over a wide area and will also release a great quantity of fine spray. In cold climates, freezing may intensify the spray problem and cause physical damage or operating difficulties at adjacent electrical facilities, as well as to buildings and vegetation.
2. On installations where the diverging jet is confined, it is difficult to entirely eliminate backsplash along the valve body. In freezing climates, there is danger that the backsplash may freeze on the valve body or operating mechanism and make operation difficult or impossible.

Valve Installation at Santa Felicia Dam

Where site conditions are favorable and operating requirements are not vere, the outlet structure can be simple and inexpensive. An example of an

outlet structure designed for moderate conditions is that for Santa Felicia Dam. This earth dam, 1400 feet long and 200 feet high above streambed, w constructed in 1955 on Piru Creek by the United Water Conservation Distri of Ventura County, California.

Santa Felicia Dam provides storage of surface waters in a coordinated surface and underground water supply system. Releases from the reservoi percolate into the streambed and into spreading grounds downstream. Ope tion of the outlet works is intermittent, and neither frequent nor rapid chan in discharge rate are required. Furthermore, the site is readily accessibl during all seasons of the year for both operation and maintenance. Simple hand operation of the discharge valves was deemed to be adequate for this installation.

Weather conditions at the site are moderate. While there are freezing temperatures during some winter nights, daytime temperatures are consis tently above freezing, and problems of freezing spray are negligible. A tot ly unconfined jet would not have caused a serious spray problem. Some mi operating inconvenience and a small additional evaporation loss would have been the only appreciable disadvantages to an unconfined jet.

The foundation at Santa Felicia consists of relatively soft interbedded sandstones and shales overlain by erodable streambed sands and gravels. was therefore considered advisable to provide a nominal stilling basin to p vent any substantial erosion.

Figure 2 shows a section of the Santa Felicia outlet works. The installa tion features a relatively simple outlet structure, housing two 36-inch hand operated fixed-dispersion cone valves manufactured by the Pelton Division Baldwin, Lima, Hamilton under the trade name of Pelton Hollow Cone Valv The maximum discharge rate is about 880 cfs under a gross head of 187 fe Operation has shown this installation to be effective in adequately preventi erosion.

Valve Installation at Vermilion Dam

Site Conditions and Operating Requirements

Vermilion Dam, completed and put into operation in late 1954 by the Southern California Edison Company, creates a 125,000 acre-foot reservoi store runoff from winter floods and spring snowmelt for later release to d stream hydroelectric plants. The dam is located at elevation 7500 feet on Mono Creek, a tributary of the upper San Joaquin River, in the Sierra Neva of California.

The earth dam is about 4200 feet long and 160 feet high above streambe It is a zoned type embankment constructed on a foundation of hetrogeneous glacial and alluvial deposits overlying bedrock to a depth of about 150 to 2 feet.

The outlet works consists of a submerged intake structure with a hydra cally operated shut-off gate discharging into a 54-inch steel pipe, which is freely supported inside an 8-foot diameter horseshoe concrete conduit ex- tending through the base of the dam. A 48-inch free-discharge regulating valve is housed in an outlet structure located at the downstream toe of the dam. The outlet works discharges into a channel excavated into relatively fine grained glacial and alluvial deposits. Because of these conditions, it necessary to control the outlet structure discharge to avoid any possibility damaging erosion of the embankment foundation.

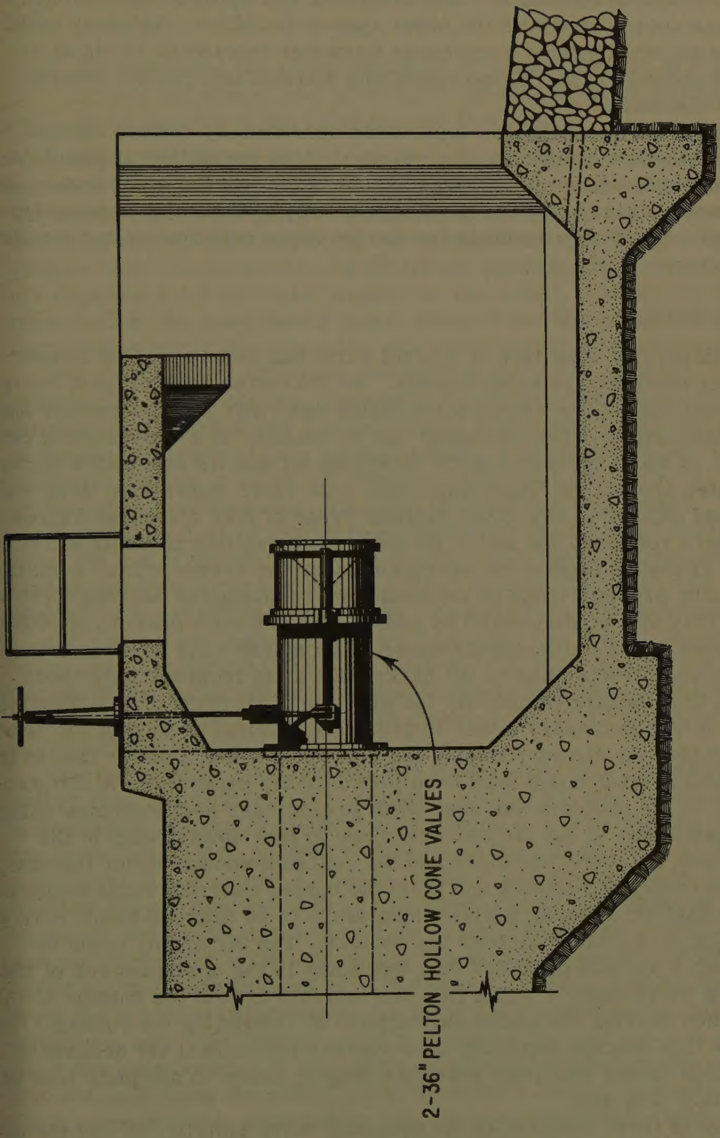


FIGURE 2

SANTA FELICIA DAM

CROSS SECTION THROUGH OUTLET STRUCTURE

Winter weather at this site in the High Sierra is severe. Temperatures may drop to 20 degrees below zero, and frequently stay below freezing for many days. Accordingly, it was recognized that the formation of ice from leakage, backsplash, and spray from the valve discharge might be a critical problem.

Vermilion Dam is about 30 miles by mountain road from the system operating headquarters at Big Creek, and is normally unattended during operation. During the winter, the site can be reached only by helicopter or snow cat. Continuous operation of the reservoir is required, since changes in release are coordinated with the other system facilities. Releases must be regulated accurately from a minimum fishwater release of 10 cfs to a maximum of about 700 cfs. These conditions dictated full remote control operation of the valve.

In summary, the necessity for dependable operation by remote control under severe weather conditions, the required close regulation of small releases, the difficult access for emergency operation and for maintenance, an the importance of dissipating the discharge energy to minimize erosion imposed a severe set of requirements for design of the outlet valve and its dis charge structure.

Preliminary Design

A fixed-dispersion cone type of control valve has some apparent advantages to meet these severe requirements. Its characteristics of good energ dissipation and simple construction are highly desirable for this remote and unattended installation. The discharge can be confined in a conventional out let structure to avoid problems of the diverging jet and its associated spray. Cost estimates showed the fixed-dispersion cone valve installation to be sub stantially less expensive than other suitable types of free discharge valves. However, there remained the major problem of controlling freezing and icin of the valve to assure dependable operation under the severe winter conditio

The freezing problem could be decreased by reducing the backsplash to a minimum. This was accomplished by adding a splash cone made to specific tions suggested by the valve manufacturer. The splash cone surrounds the diverging jet and directs most of the backsplash away from the valve body.

However, to eliminate completely the problem of freezing of any remaini backsplash, it was necessary either to provide electrical heating of the valv sleeve, or to heat the air surrounding the sleeve. Without a power line to th site, electrical heating would have been prohibitively expensive. A satisfactory solution to the problem was seen when preliminary heat exchange calcu lations indicated that, if the air could be made to flow downstream in the annulus between the steel penstocks and the concrete conduit under the dam, heat transfer from the penstock and from the buried concrete walls would be sufficient to heat air from an initial temperature of 20 degrees below zero t above freezing as it passed through the annulus. The air supply could be routed through a 12-inch diameter duct extending to the upstream end of the conduit, from which point it would flow downstream around the outside of the steel pipe, thus serving the additional purpose of ventilating the conduit. Th feasibility of this concept depended upon whether the natural air demand of the valve would create sufficient pressure drop to cause an adequate flow of air through this long path.

The design of these features of the proposed valve installation and the de termination of the most suitable form of discharge structure to minimize

spray and erosion were too complex to permit satisfactory solution by analytical methods. It was therefore decided to utilize a hydraulic model study.

Hydraulic Model Study

As generally indicated in the preceding discussion, there were two principal purposes of the model study:

1. To determine the most suitable and economical discharge structure to minimize spray and backsplash, and to prevent erosion in the outlet channel.

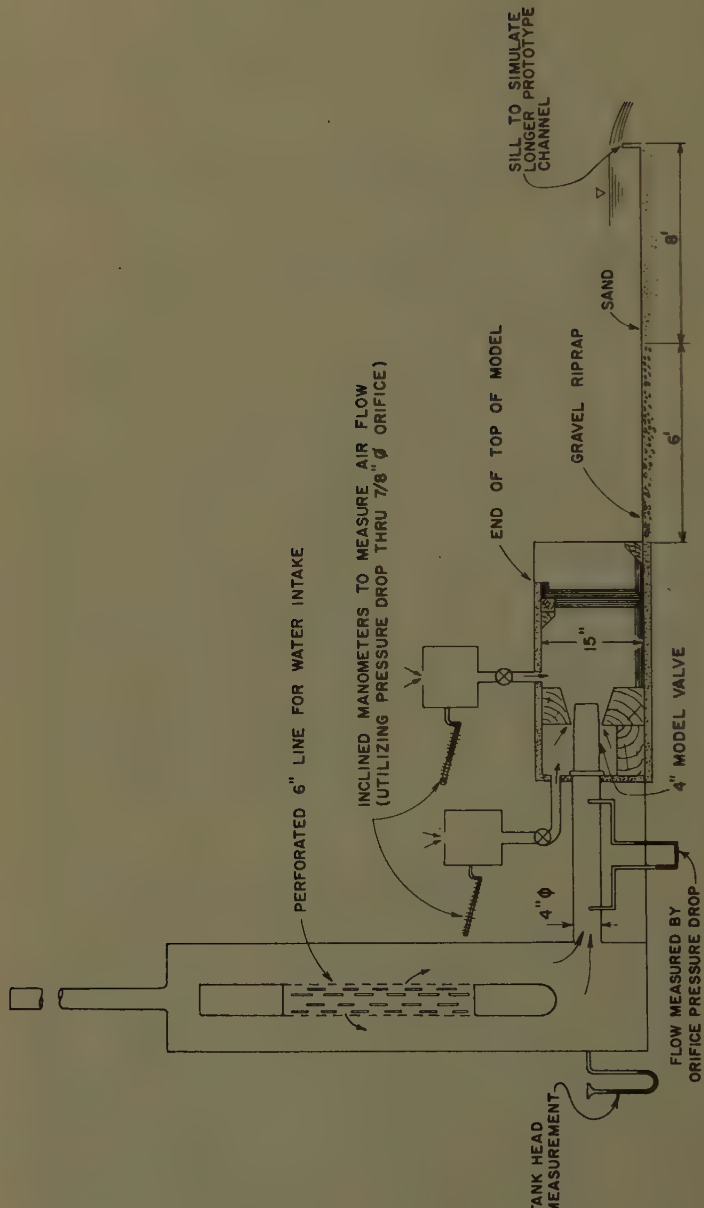
2. To determine the characteristics of the air demand of the valve in this type of structure, and investigate the feasibility of utilizing this air demand to protect the valve against freezing.

A schematic diagram of the model arrangement for the Vermilion outlet works is shown on Figure 3. A 4-inch operating model of a Howell-Bunger valve was made available for the study by S. Morgan Smith Company. The selection of a 48-inch prototype valve suggested a convenient model scale of 4:48 or 1:12. Consequently the maximum prototype discharge of about 700 cfs required a 1.5 cfs water supply for the model. The model tests were conducted at the Long Beach Steam Plant of the Southern California Edison Company, where adequate space and water supply were available.

The model outlet structure, consisting of a valve chamber, a diverging splash cone, and a stilling chamber with various interchangeable baffles, was fabricated of transparent Lucite to allow visual inspection and photographing of the flow characteristics during testing. A diverging splash cone, having a divergence angle of 20.55 degrees from the valve centerline as recommended by the valve manufacturer, was used to minimize backsplash. The equivalent of 70 prototype feet of riprap lined discharge channel was provided downstream from the end of the stilling basin to determine the effectiveness of the structure in minimizing erosion. In addition, an adjustable sill was provided at the downstream end of the model outlet channel to control tailwater.

Air flows were measured at two locations, one measuring air flow into the valve chamber behind the splash cone and the other measuring air flow into the top of the outlet chamber immediately downstream from the splash cone. Inclined manometers were used to measure the small differential air pressures across orifices.

The information desired from the model study was primarily qualitative rather than quantitative. For example, although it was necessary to know the approximate amount of discharge represented by a particular valve setting, the model was not to be used to determine the design value of the discharge coefficient of the valve. Likewise, while it was necessary to determine whether the air demand of the valve would create a substantial differential air pressure and cause an adequate flow of air through the planned facilities, it was not necessary to determine precise values of the air demand or the reduced air pressure created by this demand. In fact, such values determined from the model would probably not be directly applicable to the prototype structure, since the details of the installation had not yet been determined and were not incorporated in the model. Although a considerable amount of data was obtained from the model tests, these data are not presented in this paper since more reliable information was obtained later from tests on the completed prototype structure. The information obtained from the model tests



VERMILION DAM

DIAGRAM OF MODEL VALVE INSTALLATION

FIGURE 3

agreed in general with that determined from the prototype tests, and was sufficiently accurate to serve as a basis for design.

The model tests confirmed the feasibility of utilizing the air demand of the valve to prevent possible freezing and to provide incidental ventilation of the outlet conduit. It was found that the valve would develop an adequate reduced air pressure when the air supply was shut off, and would pull a large quantity of air, if air were available.

It was found also that cutting off the air supply from the ports downstream from the splash cone had no measurable effect on the air demand upstream. Further, the tests showed that shutting off the air supply entirely did not cause significant changes in the discharge characteristics of the valve or in the flow pattern in the discharge chamber or stilling pool. With all upstream sources of air supply shut off, the valve apparently obtains its required air through the downstream spray curtain.

The results of the model study were entirely satisfactory. Figure 4 shows a centerline section of the Vermilion valve house and discharge structure as designed. This relatively simple structure proved quite effective in adequately controlling both the spray and erosion. A photograph of the final model discharging a prototype flow of about 700 cfs is shown on Figure 5. The good conformity with prototype performance is shown on the same figure by a comparable photograph of the completed structure when discharging about 585 cfs.

Prototype Tests

Following completion of the project, and partial filling of the reservoir to provide adequate head on the valve for representative tests, Bechtel engineers, in cooperation with representatives of Southern California Edison Company and S. Morgan Smith Company, performed a series of prototype tests. There were three principal purposes of the tests:

1. To verify the theoretically computed discharge rating and establish an accurate discharge curve for use in reservoir operation.

2. To investigate the various factors affecting the air demand and to determine the probable effectiveness of the air supply features in controlling icing.

3. To check the effectiveness of the outlet structure in controlling erosion in the downstream channel.

Tests were run for five different settings of the valve: 97.5%, 70%, 40%, 20%, and 5% open. At each setting, observations were made for five different conditions of air supply. The following data were obtained for each test condition, as appropriate:

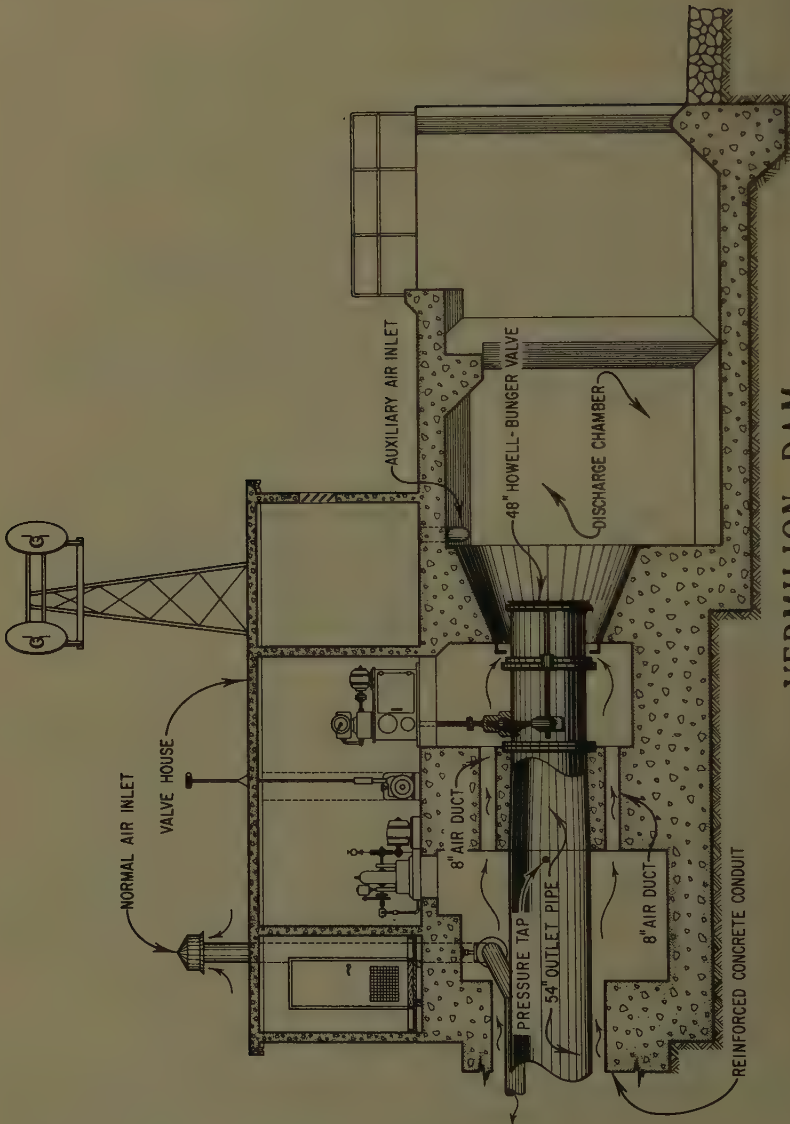
1. Reservoir elevation.

2. Gage pressure at a pressure tap 6.5 feet.

3. Stream-gage measurements of the valve discharge in the channel downstream from the outlet structure.

4. Air velocity measurements at the various intake ports for supplying air to the valve.

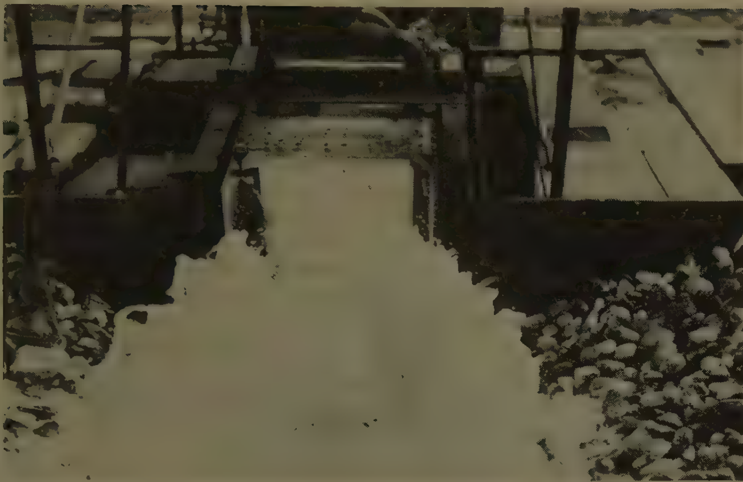
5. Reduced air pressure in the valve pit, below atmospheric pressure.



VERMILION DAM

SECTION THROUGH OUTLET STRUCTURE

FIGURE 4



MODEL DISCHARGING AT FULL GATE
RESERVOIR AT 136 FT. STATIC HEAD
Q = 700 CFS

PROTOTYPE DISCHARGING AT FULL GATE
RESERVOIR AT 100 FT. STATIC HEAD
Q = 585 CFS

FIGURE 5

6. Photographs of the backsplash, if any, in the valve pit.

7. Photographs of the discharge from the outlet structure.

Figure 6 shows a graph comparing the coefficient of discharge determined from the test measurements with the values published by the valve manufacturer. The coefficient is defined as $C_D = Q/A\sqrt{2gH}$, where H is the total of pressure and velocity head at the valve. It was assumed that the losses in the 6.5 feet of pipe from the pressure tap to the valve were negligible. It will be noted that the curves agree closely. The measured coefficient of 0.86 for maximum opening (97.5% stroke) compares with the manufacturer's value of 0.85, for example. This small different is believed to be within the limits of accuracy of the Vermilion tests, and to confirm the manufacturer's values.

The Vermilion installation has three different sources of air supply: (1) air admitted upstream from the valve which flows downstream through the annular opening between the valve and the splash cone; (2) air admitted into the top of the outlet structure through ports at the downstream end of the splash cone; and (3) unmeasured and uncontrolled amounts of air entering the spray curtain through the open downstream end of the outlet structure.

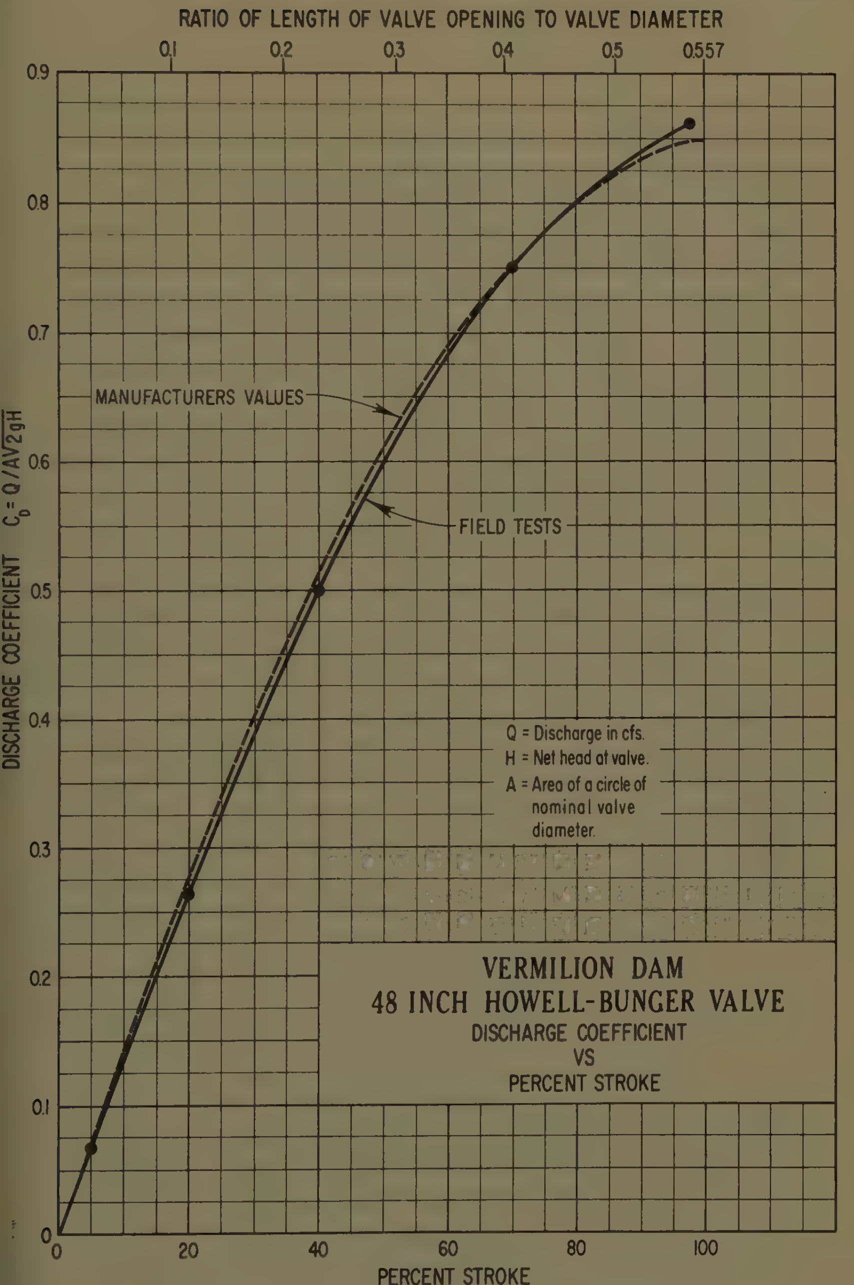
Air admitted upstream from the valve was measured by means of a torsion type anemometer, all readings being corrected for the reduced density of air at the 7500-foot elevation. Velocities through the ports supplying air to the upstream side of the valve ranged up to about 20 feet per second. When the blind flanges were removed from the 12-inch diameter auxiliary air inlets in the top of the outlet structure, air velocities through these openings were as high as 150 feet per second. Since this is far in excess of the capacity of the torsion type anemometer, a pitot tube and manometer were used to measure the high velocities.

Figures 7 and 8 are graphs in two different forms showing the relationship between air discharge and water discharge for the controllable air supply to the valve, and Figure 9 is a plot of the reduction of air pressure in the valve pit below atmospheric pressure when all controllable air supply is cut off. It should be noted that none of these curves depicts the actual operating condition, as the air discharge curves on Figures 7 and 8 show the maximum air demand with minimum restriction of the air supply (i.e., minimum pressure drop) and the air pressure curve on Figure 9 shows the maximum pressure reduction with complete restriction of the air supply.

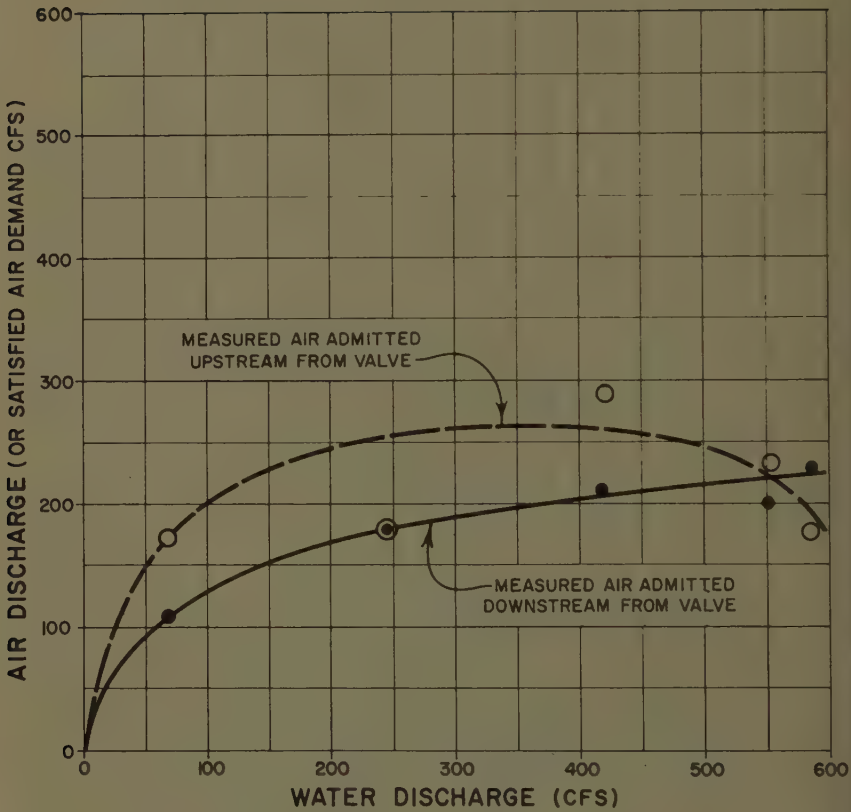
The air supply facilities for the valve provide better ventilation than expected in the conduit beneath the dam. The air flow back through the conduit not only is adequately heated but essentially eliminates condensation on the 54-inch outlet pipe and keeps the air fresh in the buried conduit.

The prototype valve tests indicated the amount of backsplash to be dependent more upon valve stroke than upon the controllable conditions of air supply. The amount of backsplash ranged from essentially none at 97.5% and 70% gate openings, to a maximum at about 20% gate opening, then decreasing gradually to a still appreciable amount at 5% gate opening. At 40% gate, water lapped back into the valve pit, over and around the valve. At 20% gate, the backsplash, though mild in velocity, was sufficient to splash observers in the valve pit.

In addition to the above prototype tests, a series of temperature readings were made one night when the outside temperature dropped to 20 degrees below zero. Minimum indicating thermometers showed the following minimum temperatures at three pertinent locations: 38 degrees in the valve pit


RATIO OF LENGTH OF VALVE OPENING TO VALVE DIAMETER
0.1
0.2
0.3
0.4
0.5
0.557
DISCHARGE COEFFICIENT $C_D = Q/A\sqrt{2gH}$
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
MANUFACTURERS VALUES
FIELD TESTS
Q = Discharge in cfs.
H = Net head at valve.
A = Area of a circle of nominal valve diameter.
VERMILION DAM
48 INCH HOWELL-BUNGER VALVE
DISCHARGE COEFFICIENT
VS
PERCENT STROKE
0
20
40
60
80
100
PERCENT STROKE

FIGURE

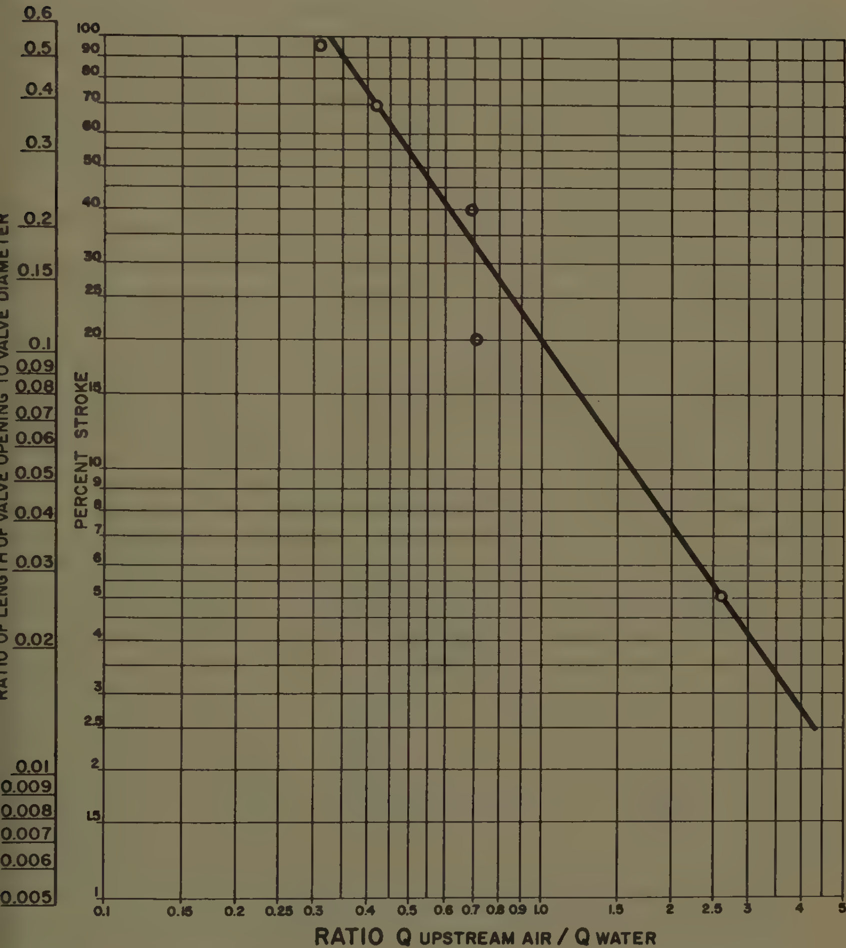


VERMILION DAM

48 IN. FIXED-DISPERSION CONE VALVE

AIR DISCHARGE VS WATER DISCHARGE

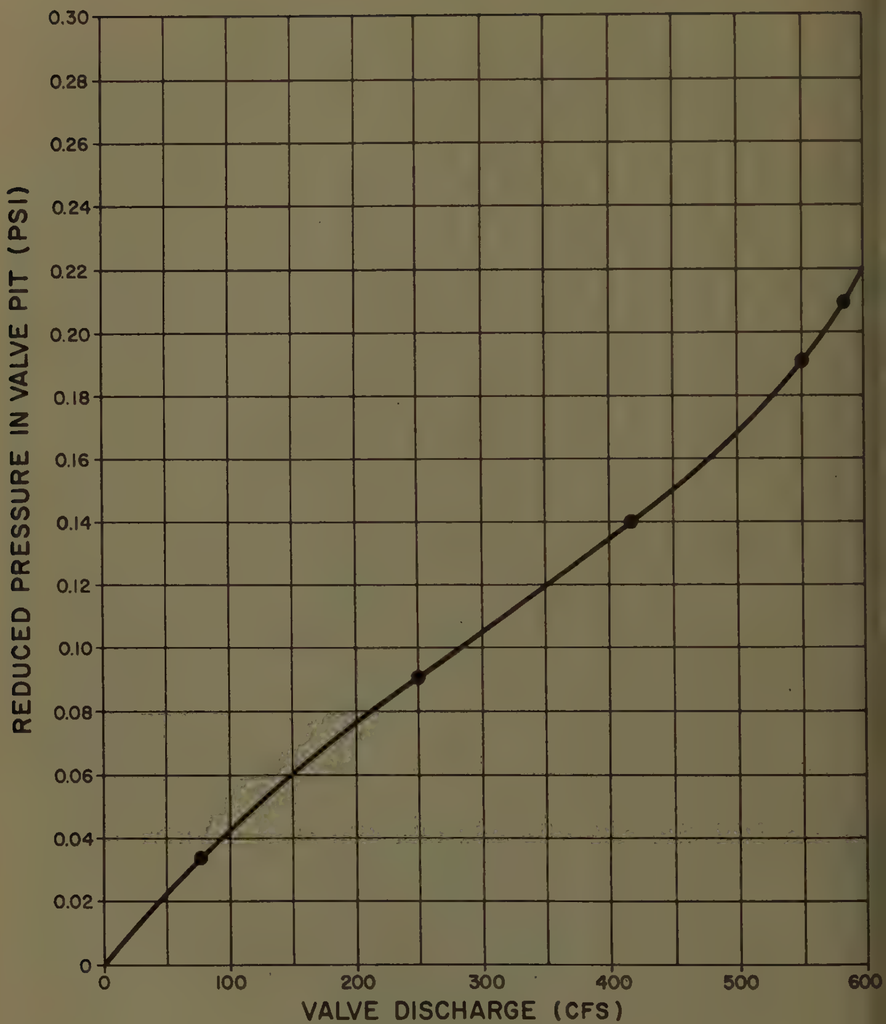
FIGURE 7



VERMILION DAM

VARIATION OF AIR-WATER DISCHARGE RATIO
WITH SLEEVE POSITION

FIGURE 8



VERMILION DAM

48-IN. FIXED-DISPERSION CONE VALVE
PLOT OF REDUCED PRESSURE IN VALVE PIT VS DISCHARGE
ALL CONTROLLABLE AIR CUT OFF

FIGURE 9

acent to the valve sleeve, 38 degrees in the conduit upstream from the ve anchor block, and 35 degrees in the valve house adjacent to the valve erator. During the night, two small patches of ice formed on the walls of structure downstream of the discharging jet. The maximum thickness of s ice was about two inches and the areas were about 8 inches wide and 5 t long. A thin layer of ice and frost also formed on the outside of the 12-h air duct for about the first 50 feet of its length in the conduit beneath the n.

Careful observations during high valve discharges have indicated no visible vement of material in the channel downstream from the outlet structure, l no noticeable erosion has occurred after two years of operation.

Valve Installation at Portal Powerhouse

Subsequent to the construction of Vermilion Dam and the performance of prototype tests, another outlet structure was designed for Southern Cali-nia Edison Company to house a 90-inch bypass valve at Portal Powerhouse Huntington Lake, California. This powerhouse is at elevation 7000 feet at end of a 13-mile long, 12-foot diameter tunnel.

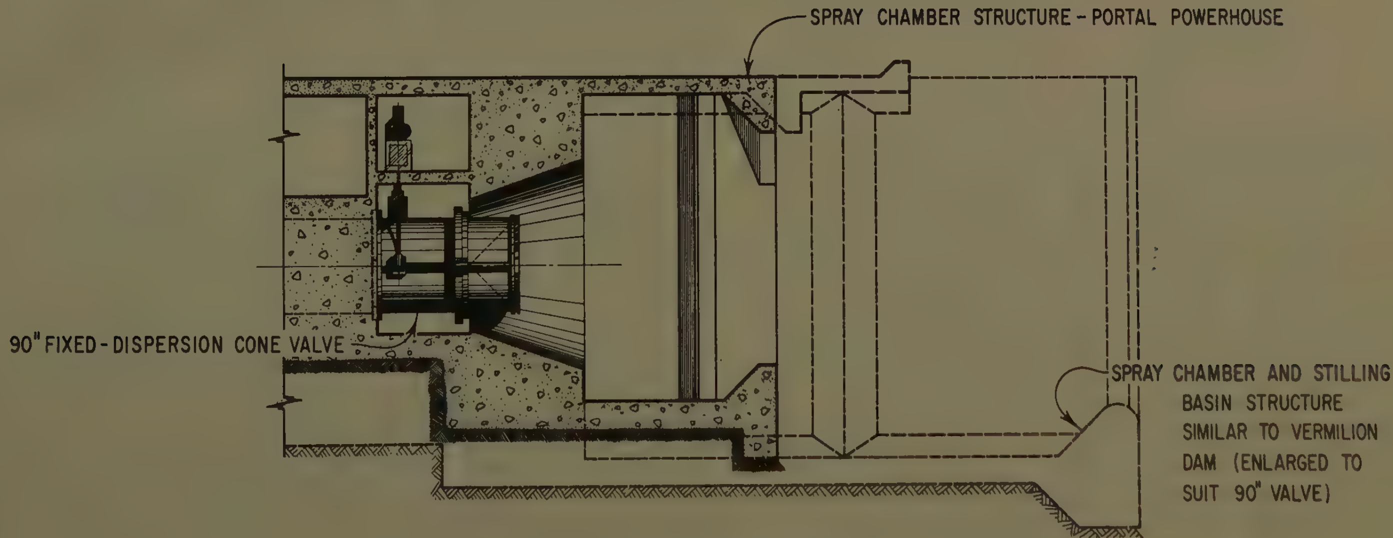
Portal Powerhouse is more accessible than Vermilion. However, like rmilion, the installation is designed for unattended operation. The tem-rature at Portal is as severe as that at Vermilion, but electric power is ilable for heating the valve sleeve to prevent freezing. The decomposed nite foundation at the site is far superior to the foundation at Vermilion. vertheless, control of the spray to eliminate condensation and freezing was sidered necessary.

Because of the satisfactory Vermilion experience, it was decided to use a -inch fixed-dispersion cone valve. The designs provided an outlet struc-e geometrically and hydraulically similar to that at Vermilion but without tilling basin, since none was required for this foundation. This resulted in onsiderable saving, as can be seen from the comparative sections of the rmilion and Portal installations shown on Figure 10.

On the basis of the Vermilion prototype tests, it was decided to design the rtal installation with no air supply to the valve either upstream or down-eam of the spray curtain. Provision was made, however, by arrangement access hatches and by leaving a blockout in the outlet structure wall, for ure provision of air in the event that hydraulic disturbances developed be-ise of the greater vacuum which this valve was expected to create.

When the valve was placed in operation in 1956, air pressure readings re made in the valve pit. For a valve discharge of approximately 1300 cfs ler a 200-foot static head (about 70 percent of maximum discharge), a nega-e pressure of about 1.8 psi was produced in the valve pit. Fortunately, as esult of the Vermilion tests, a sizeable vacuum had been anticipated. ong air tight hatches were provided between the valve pit and the power-ise, and the spray chamber walls were designed to withstand a considerable ;ative pressure.

To date, the Portal installation has operated throughout one winter and has nained entirely free of freezing in the immediate vicinity of the outlet ucture. Some frost has been noted in the downstream area adjacent to the let channel, where a fog is created when the discharging water mixes with colder atmosphere. Figure 11 shows the Portal installation discharging



PORTAL POWERHOUSE AND VERMILION DAM

COMPARATIVE CROSS SECTIONS OF OUTLET STRUCTURES

FIGURE 10

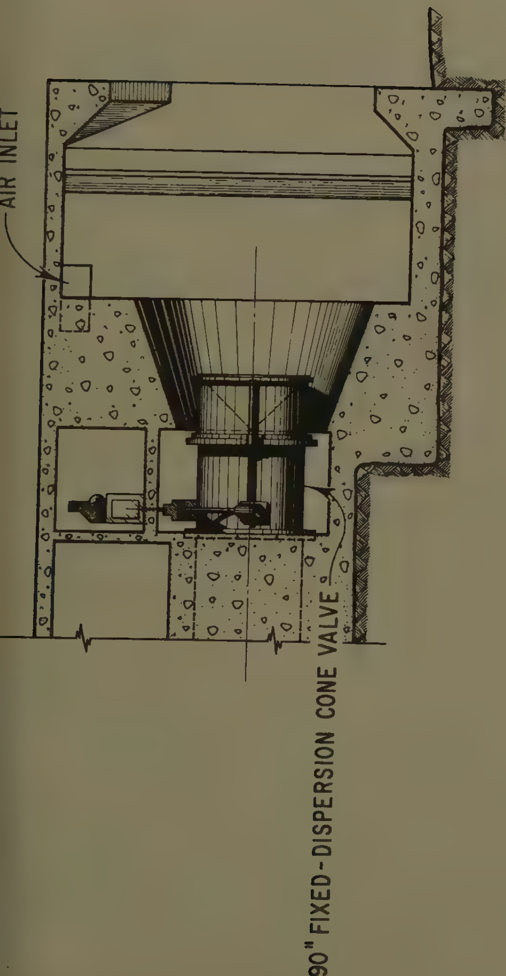


FIGURE 11

PORTAL POWERHOUSE

CROSS SECTION THROUGH DISCHARGE CHAMBER

through the outlet chamber. Note the completely dry concrete around the opening.

Cavitation has not been a problem at any of the installations.

CONCLUSIONS

The experiences cited above indicate that by individually designing outle structures to suit particular site and operating requirements, fixed-disper sion cone valves may be adapted for successful and economical use under wide range of conditions. At Vermilion Dam and Portal Powerhouse, the problems of spray and freezing, normally attendant to a fixed-dispersion c valve, were satisfactorily solved by a carefully designed outlet structure, thus obviating the expense of a more complicated valve and structure.

ACKNOWLEDGMENTS

The authors are indebted to the engineering and operating personnel of Southern California Edison Company who made the Vermilion model and prototype tests possible; also to Martin M. Loewenthal, J.M. ASCE, who pe sonally directed the model tests; and the S. Morgan Smith Company, who p vided the model Howell-Bunger valve.

The authors also gratefully acknowledge the contributions of James W. Ball, Chairman of the Committee on Publications, for his helpful suggestio and of Ralph E. Bolles, A.M. ASCE, and Howard M. Whitelaw, who rendere valuable assistance in the preparation of this paper.

The Pelton Division of the Baldwin, Lima, Hamilton Company manufact the Santa Felicia valves, and S. Morgan Smith Company manufactured the valves installed at Vermilion Dam and Portal Powerhouse.

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Proceedings of the American Society of Civil Engineers

HURRICANE PROTECTION PLANNING IN NEW ENGLAND[a]

John B. McAleer,[1] A.M. ASCE and George E. Townsend,[2] A.M. ASCE
(Proc. Paper 1726)

ABSTRACT

Hurricane Carol in August 1954 caused damages of $300,000,000 and the ss of 60 lives in New England, and triggered authorization of a hurricane rvey of the Atlantic and Gulf Coasts. This paper describes some of the sic data and engineering methods used in the relatively new field of hurri-ne protection planning in the New England area, and especially for New edford and Narragansett Bay.

General Problem and Scope

After Hurricane Carol in August 1954 which caused damages of about 00,000,000 and loss of 60 lives in the Northeast, the Congress authorized 15 June 1955 an examination and survey of the eastern and southern sea-ard of the Atlantic and Gulf coasts to be made under the direction of the hief of Engineers, in cooperation with the Department of Commerce, mainly e Weather Bureau, and other Federal agencies concerned with hurricanes. Congress directed that the survey should include:

1. the securing of data on the behavior and frequency of hurricanes,
2. determination of methods of forecasting and improved warning services, and

te: Discussion open until January 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1726 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. HY 4, August, 1958.
Presented at meeting of ASCE Hydr. Conf., Cambridge, Mass., August, 1957.
Chf. Hurricane Protection Unit, Corps of Engrs., U. S. Army, New England Div., Boston, Mass.
Hydro. Engr., Corps of Engrs., U. S. Army, New England Div., Boston, Mass.

3. possible means of preventing loss of life and damage to property, wit due consideration of the economics of proposed breakwaters, seawall dikes, dams and other structures, warning services or other measur which might be required.

The survey was first initiated in New England where particularly severe damage from tidal-flooding occurred in 1954. The New England Division, Corps of Engineers, now under the direction of Brigadier General Alden K. Sibley, has completed interim reports recommending construction of prote tive works in two areas: (1) Narragansett Bay area, Rhode Island and Massachusetts, and (2) New Bedford-Fairhaven area, Massachusetts. Thes projects are presently under consideration by Congress. (See Plate 1) Son of the first principles of hurricane protection planning and the engineering plication of scientific studies has been directed to the problems of tidal-flo protection in these damage centers.

Methods of Protection

For protection against hurricane tidal-flooding there are a wide variety methods which vary in effectiveness, cost and application. Each method ha its limitations and is likely to create certain problems and conflicts with existing development of the area. The principal methods are as follows:

1. Hurricane Warning and Emergency Flood Mobilization Measures. O method of reducing loss of life and property is to forecast the movement of hurricane, assure appropriate warnings, and take necessary emergency m sures for evacuation of the flood areas. As part of its responsibility for in proved weather services in connection with major storms and hurricanes, Weather Bureau has established a "severe weather" network along the Atla tic Coast, using powerful radarscopes. These activities represent one of t most important advances in hurricane protection.

Even though a warning system may be as elaborate and reliable as poss ble, it does not always provide sufficient time for adequate precautions. F instance, the hurricane of 1938, which was at one time reported stalled off Cape Hatteras, North Carolina, swept over New England almost unannounc only eight hours later.

Emergency work must of necessity start very early as considerable tim is required for the preparation of homes, buildings, goods and other prope such as boarding up and sandbagging lower floors and windows, removing goods and equipment to higher levels, and evacuating low-lying areas. Sm vessels must be pulled ashore and large craft moved to protected areas. Vehicles are driven out of the flood area while aircraft are flown west.

After the temporary preventive measures have been initiated the threat may not materialize. The hurricane alerts and near misses that result in "scares" only may seriously interfere with the normal activities of the af- fected residents and mean undue hardship and great economic loss.

2. Revision of Zoning Regulations and Building Codes. The limitations hurricane warnings and emergency measures logically leads to considerat of more permanent measures, such as the permanent relocation of goods a equipment to higher floor levels, relocation out of the flood area entirely, of more substantial construction, to resist the destructive forces of high water and waves. Zoning regulations and revision of building codes are a

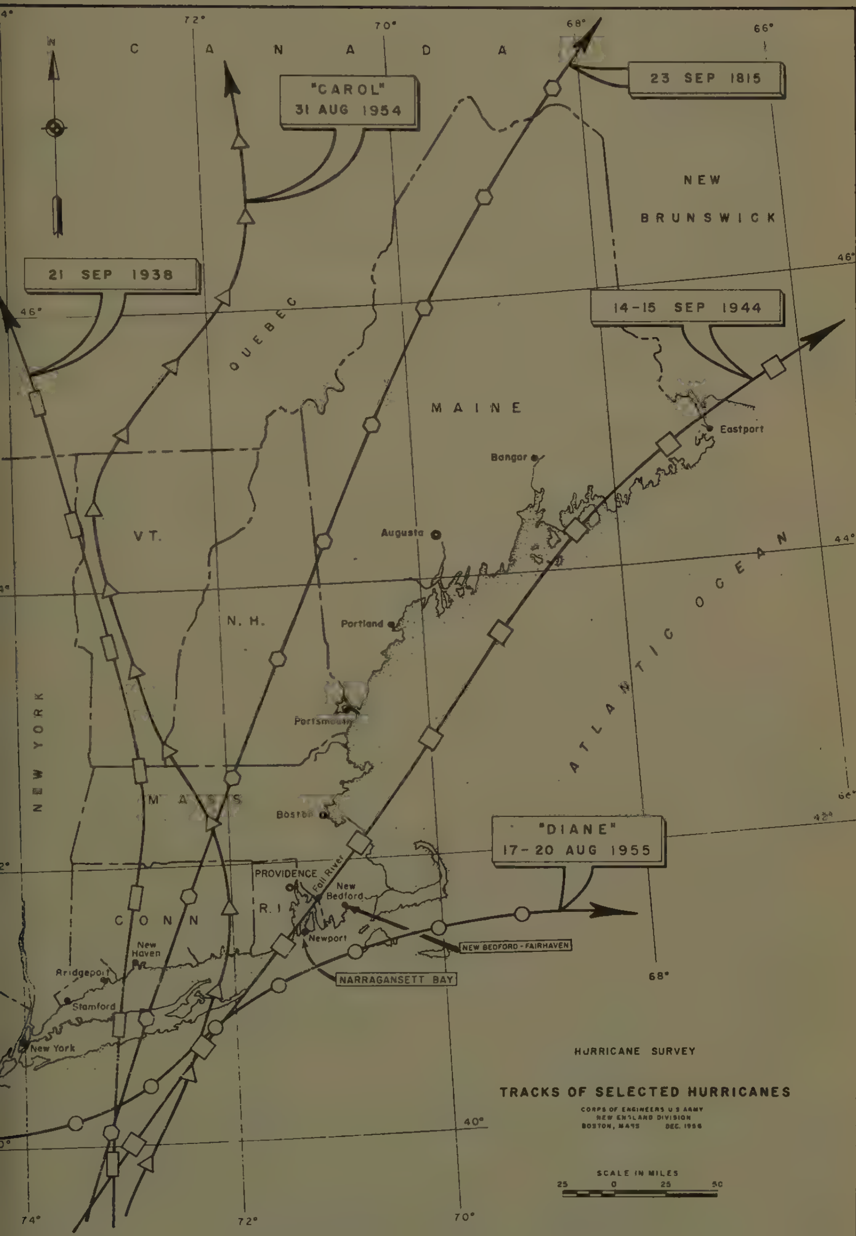

C A N A D A
"CAROL"
31 AUG 1954
23 SEP 1815
21 SEP 1938
14-15 SEP 1944
NEW BRUNSWICK
QUEBEC
MAINE
VT.
N. H.
NEW YORK
MASS
CONN
R. I.
Eastport
Bangor
Augusta
Portland
Portsmouth
Boston
PROVIDENCE
Fall River
New Bedford
Newport
New Haven
Bridgeport
Stamford
New York
ATLANTIC OCEAN
"DIANE"
17-20 AUG 1955
NEW BEDFORD - FAIRHAVEN
NARRAGANSETT BAY
HURRICANE SURVEY
TRACKS OF SELECTED HURRICANES
CORPS OF ENGINEERS U S ARMY
NEW ENGLAND DIVISION
BOSTON, MASS DEC. 1956
SCALE IN MILES
25 0 25 50

PLATE I

solution in that much of the loss of life in hurricanes occurs because cottag homes and industries should never have been located on the coastal flood plain.

State and local governments have proposed adoption of zoning restriction to prevent construction in critical areas, and revision of building codes to r quire more rugged structures for areas where buildings were demolished b the storm tide. Minimum elevations have been specified for the floor level of new structures. For concentration of homes, commercial establishment and industries, such measures have met with strong opposition because of t great investment in property and the prospective loss to property owners a municipalities.

3. Local Protection. Three general classes of local protection are considered:

a. Individual measures. Since Hurricane Carol, a number of the larg industries and business establishments in the flood area have installed permanent or semi-permanent measures to protect their physical plants again hurricane flooding. These measures include:

1. construction of flood-proof structures,
2. construction of flood walls around individual properties,
3. permanent closure of windows and other openings exposed to fl waters,
4. installation of valves or gates to prevent backup in pipe lines,
5. installation of pumps to control seepage in interior drainage, a
6. changes in the utilization of space subject to flooding.

Many of these individual measures are practical and economical. Howe some of the work has been done without adequate factors of safety against overtopping, uplift, or seepage and the work, as yet untested, is of uncertai effectiveness. In some cases the construction is costly not only in materia and labor, but also in interfering with efficient use of space and reducing p ductivity. Individual measures tend to be uneconomical as compared to alternative flood protection of a number of properties in a single unit.

b. Low walls and bulkheads to protect against erosion. Many owners shorefront properties have constructed concrete walls or bulkheads to protect against erosion and reduce wave damage. These structures are highly practical for individual shorefront properties and will frequently prevent wholesale destruction of buildings and improvements. In no sense can they relied upon to provide adequate protection against tidal-flooding. They so times give a false sense of security.

c. Local dikes, walls and barriers. In many locations a group of cottages, industries, or a commercial area can be protected against tidal-floo ing by means of walls and dikes. The high cost of such construction will normally require a concentration of valuable properties. Restoring or rai the natural sand dunes will provide protection in some areas. Beach erosi control measures are likely to be required. To protect against hurricane floods with wave attack and wave overtopping, such construction is likely t require heights of 20 feet or more, msl. At many locations problems resu

from blocking view and access to the shore front, and the work would be objectionable to property owners. Stoplog openings are required for highways and railroads and fresh water run-off may require construction of pumping stations and storm sewers.

4. Protection against wave action by breakwaters. In exposed coastal areas where properties are subject to heavy wave attack, breakwaters are an effective method of affording protection to shorefront property and small craft. Although such construction is effective in reducing wave heights it will not reduce the still water level of tidal-flooding.

5. Large-scale tidal-flood barriers. The topography of many rivers, estuaries and bays is adapted to the construction of some type of dam or series of barriers to prevent the entry of the tidal surge into the area. Occasionally, islands and natural constrictions at the mouth of the bay and the existence of highway, railway embankments, or bridge crossings, are well adapted for use as a hurricane barrier, as at New Bedford, Massachusetts and Narragansett Bay, Rhode Island and Massachusetts, described below.

New Bedford-Fairhaven Plan of Protection

Local interests of New Bedford, Fairhaven and Acushnet, Massachusetts advanced a number of proposals for protection against hurricane tidal flooding. After analyses of seven alternate plans, a plan was recommended as promising economic protection for the survey area with minimum disruption to existing facilities.

The recommended plan for flood protection consists of three structures shown on Plate 2. The largest and most important of these is a barrier across New Bedford Harbor in the vicinity of Palmer Island. Supplemental dike and wall protection is provided in the Clark Cove area of New Bedford and in Fairhaven, to prevent flanking of the main barrier.

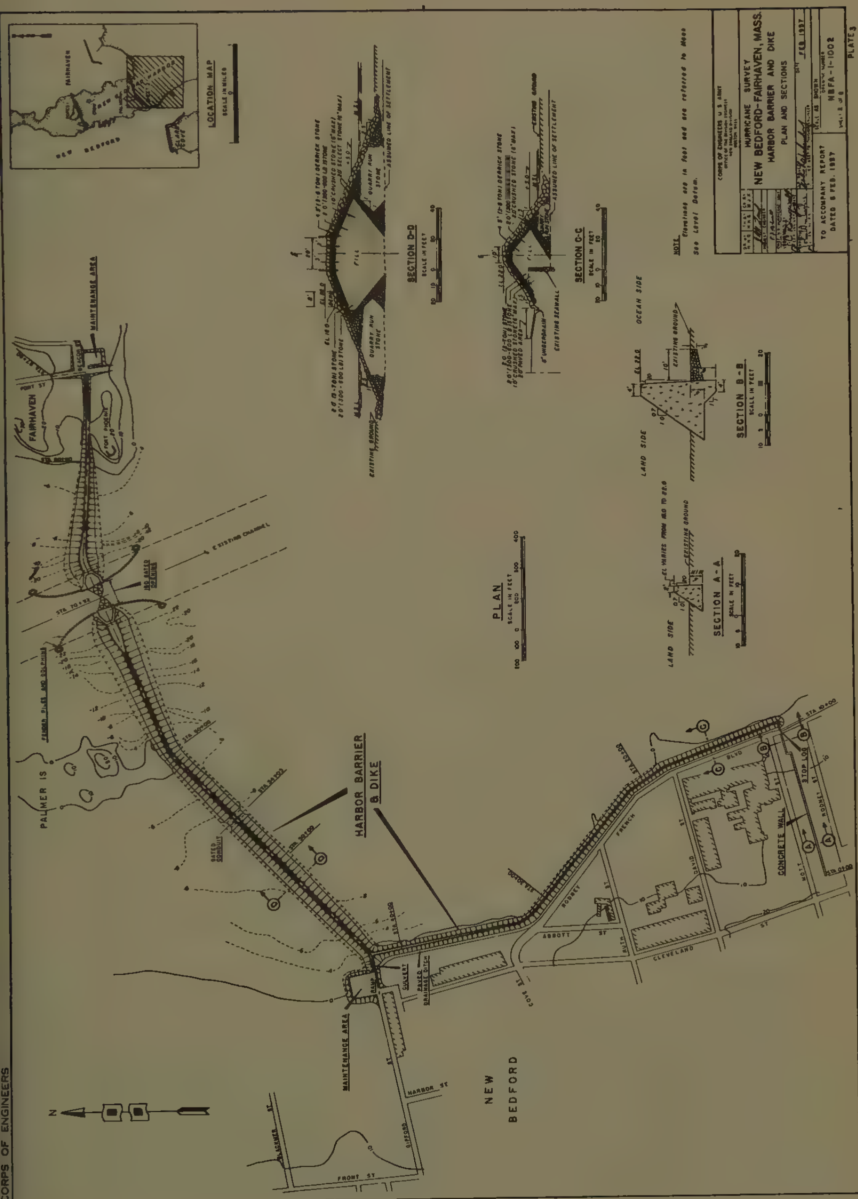
The Main Harbor barrier, shown on Plate 3, consists of 4,430 feet of earth-fill dike, with rock faces and toes, extending across the main harbor in the vicinity of Palmer Island, and tying into high ground on either side. The structure would have a top elevation of 22 feet msl, a top width of 20 feet and side slopes of 1 on 2.5. A 20-foot width was adopted to provide a required roadway along the top of the barrier. To withstand the attack of hurricane waves, 3- to 6-ton derrick stone is provided for the outer face and 2- to 3-ton stone for the inner face.

A gated opening 150-feet wide, is provided to permit the continuance of vessel traffic in and out of the harbor. Each navigation gate would have a radius of 90 feet, a central angle of approximately 60°, a total height of 61 feet, and a weight of over 500 (516) tons. (See Plate 4.) The elevation of the gate sill would be 39 feet below msl or 37.2 feet below mlw. Normally the gates would be kept in an open position with each gate set into a recess in its abutment. They would be closed only when a hurricane threatens the area with tidal flooding.

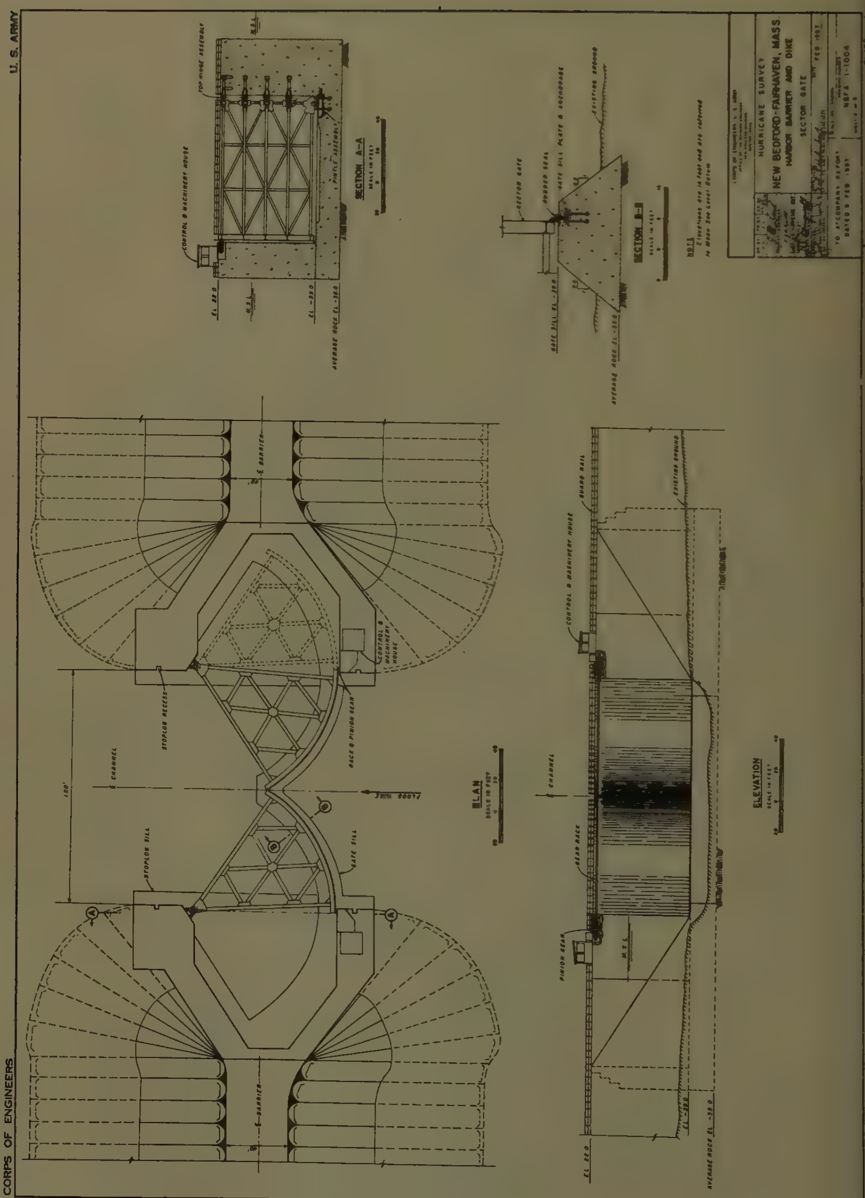
A gated conduit is provided in the section of the barrier between the New Bedford shore and Palmer Island to permit emergency emptying of the pool that would form behind the barrier when the gates in the opening are closed to navigation. It would also serve to permit a circulation of tidal flow on the west side of Palmer Island.


ACUSHNET
MATTAPOISETT
NEW BEDFORD
FAIRHAVEN
DARTMOUTH
ACUSHNET RIVER
NEW BEDFORD HARBOR
FAIRHAVEN DIKE
HARBOR BARRIER & DIKE
CLARK COVE DIKE
CLARK COVE
CLARK PT
NASKETUCKET BAY
SCONTICUT NECK
SCONTICUT PT
WEST ISLAND
APPONAGANSETT BAY
NEW BEDFORD
DARTMOUTH
PLYMOUTH CO
BRISTOL CO
BUZZARDS BAY
MUNICIPAL AIRPORT
N
LOCATION MAP
SCALE IN FEET
LEGEND
FLOODED AREA DESIGN HURRICANE EL 18.0 MSL
CORPS OF ENGINEERS, U. S. ARMY
HURRICANE SURVEY
NEW BEDFORD-FAIRHAVEN, MASS.
GENERAL PLAN
FEB 1957
SCALE AS SHOWN
TO ACCOMPANY REPORT DATED 8 FEB. 1957
NBFA- 1-1000

PLATE 2



CORPS OF ENGINEERS


U. S. ARMY
CORPS OF ENGINEERS
HURRICANE SURVEY
NEW BEDFORD-FAIRHAVEN, MASS.
HARBOR BARRIER AND DIKE
SECTOR GATE
SECTION A-A
SECTION B-B
PLAN
ELEVATION
CONTROL & MACHINERY HOUSE
STOPLOG SILL
STOPLOG RECESS
RACK & PINION GEAR
GATE SILL
GUARD RAIL
EXISTING GROUND
GEAR RACK
PINION GEAR
EL 22.0
AVERAGE ROCK
PLATE 4

A dike extension to the barrier, constructed of earth fill with rock facing, provided along Rodney French Boulevard in New Bedford for a distance of out 3,100 feet.

Narragansett Bay Plans of Protection

Soon after the 1954 hurricane, local interests requested protection of ovidence and Narragansett Bay against hurricane tidal flooding. At least plans involving 25 locations for protective barriers were proposed by pri- te engineering firms, State and Federal agencies, and interested parties. ter thorough engineering studies of the better plans, including foundation plorations and model tests, a two-unit solution of the problem (See Plate 5) s recommended.

1. For the protection of the business center of Providence, Rhode Island— nstruction of a concrete barrier and pumping station across the Providence ver at Fox Point in Providence.

2. For the general protection of Narragansett Bay—the construction of rock l barriers across the East and West Passages of Lower Narragansett Bay d a barrier across the Sakonnet River at Tiverton, subject to more detailed sign studies of the structures and their effects on navigation, pollution and heries.

Fox Point Barrier

A concrete gravity dam about 1,100 feet long would be constructed across e Providence River at Fox Point (See Plate 6). Four sluice gates would be cluded to pass the normal river flow and a pumping station for passing river w under flood conditions when the gates are closed. Reinforced concrete d walls at either end would tie into high ground.

Design of the barrier with top elevation at 22.5 feet msl allows for 3.8 feet freeboard for the design flood. A 37-foot wide deck has been set at eleva- n 12.5 feet msl for wharfage or highway use.

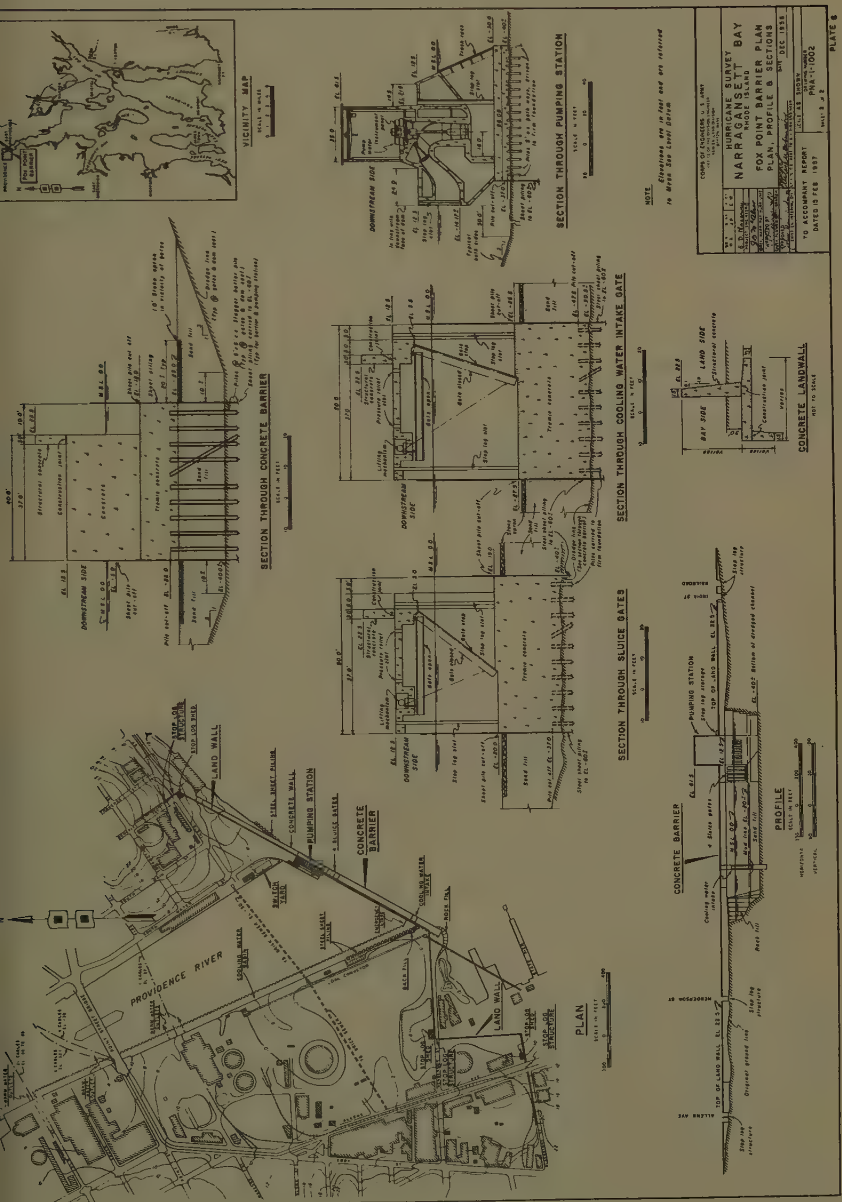
Sluice gates, each 20 feet wide by 24 feet high, would be of a drop type osing by gravity when released, to prevent entry of flood waters from the y. They would be capable of discharging all river flow, including full flood n-off at times of hurricane surge. They would also pass the normal flood d ebb tides.

The concrete barrier dam is supported on piles driven to approximately feet below mean sea level. Borings show that the foundation is of variable ts, sands, and gravels overlaid by a heavy deposit of organic silt.

The pumping station would contain five large pumps for a combined capacity 8,000 cfs at a differential head of 22 feet. During hurricane periods the ol back of the barrier would be maintained at an elevation of 3 feet below l for inflows up to 8,000 cfs. In the event of a maximum storm runoff of 00 cfs, the pumps could pass this larger rate of run-off with ponding to vation 3 feet msl.


NORTH PROVIDENCE
PAWTUCKET
SEEKONK
TAUNTON
BERKLEY
PROVIDENCE
FOX POINT
BARRIER
EAST PROVIDENCE
DIGHTON
REHOBOTH
CRANSTON
FREETOWN
PROVIDENCE RIVER
SWANSEA
BARRINGTON
WARREN
GREENE AIRPORT
APPONAUG
WARWICK
FALL RIVER
GREENWICH BAY
MT. HOPE BAY
BRISTOL
EAST GREENWICH
PATIENCE
TAUNTON RIVER
TIVERTON
TIVERTON
BARRIER
PRUDENCE ISLAND
PORTSMOUTH
WESTPORT
LEGEND
EL 18.7 (PROVIDENCE)
EL 13.0 (NEWPORT)
WICKFORD
NORTH KINGSTOWN
MIDDLETOWN
WEST BARRIER
LITTLE
COMPTON
JAMESTOWN
NEWPORT
EAST
BARRIER
SOUTH KINGSTOWN
AQUIDNECK
SAKONNET RIVER
CORPS OF ENGINEERS, U. S. ARMY
HURRICANE SURVEY
NARRAGANSETT BAY
RHODE ISLAND-MASSACHUSETTS
GENERAL PLAN
DEC. 1956
TO ACCOMPANY REPORT
DATED 15 FEB. 1957
SCALE AS SHOWN
PNA-1-1000
LOCATION MAP
NARRAGANSETT BAY
NARRAGANSETT PIER

PLATE 5


VICINITY MAP
SECTION THROUGH CONCRETE BARRIER
SECTION THROUGH PUMPING STATION
SECTION THROUGH COOLING WATER INTAKE GATE
SECTION THROUGH SLUICE GATES
CONCRETE LANDWALL
NOTE
Elevations are in feet and are referred to Mean Sea Level Datum
PLAN
PROFILE
PROVIDENCE RIVER
LAND WALL
CONCRETE WALL
PUMPING STATION
CONCRETE BARRIER
SWITCH YARD
CONCRETE BARRIER
PUMPING STATION
CORPS OF ENGINEERS U.S. ARMY
HURRICANE SURVEY
NARRAGANSETT BAY
RHODE ISLAND
FOX POINT BARRIER PLAN
PLAN, PROFILE & SECTIONS
DEC 1956
TO ACCOMPANY REPORT
DATED 15 FEB 1957
PLATE 6

Lower Bay Barriers

The East barrier (See Plate 7) would be a massive stone structure 3,200 feet long extending across the East Passage at Newport. It would be constructed with a quarry-run stone core capped and faced with heavy derrick stones. The top elevation would be of 20-foot width at an elevation of 22 feet msl with side slopes of 1 on 2. The maximum base width of the structure would be more than 700 feet in a water depth of 165 feet. The structure woul be oriented at right angles to the existing navigation channel and a navigation opening provided, 1,000 feet wide and 50 feet deep at mean low water.

Similar structures would be constructed across the West Passage at Jamestown Bridge (See Plate 8) and across the Sakonnet River at Tiverton (See Plate 9).

Survey Procedures

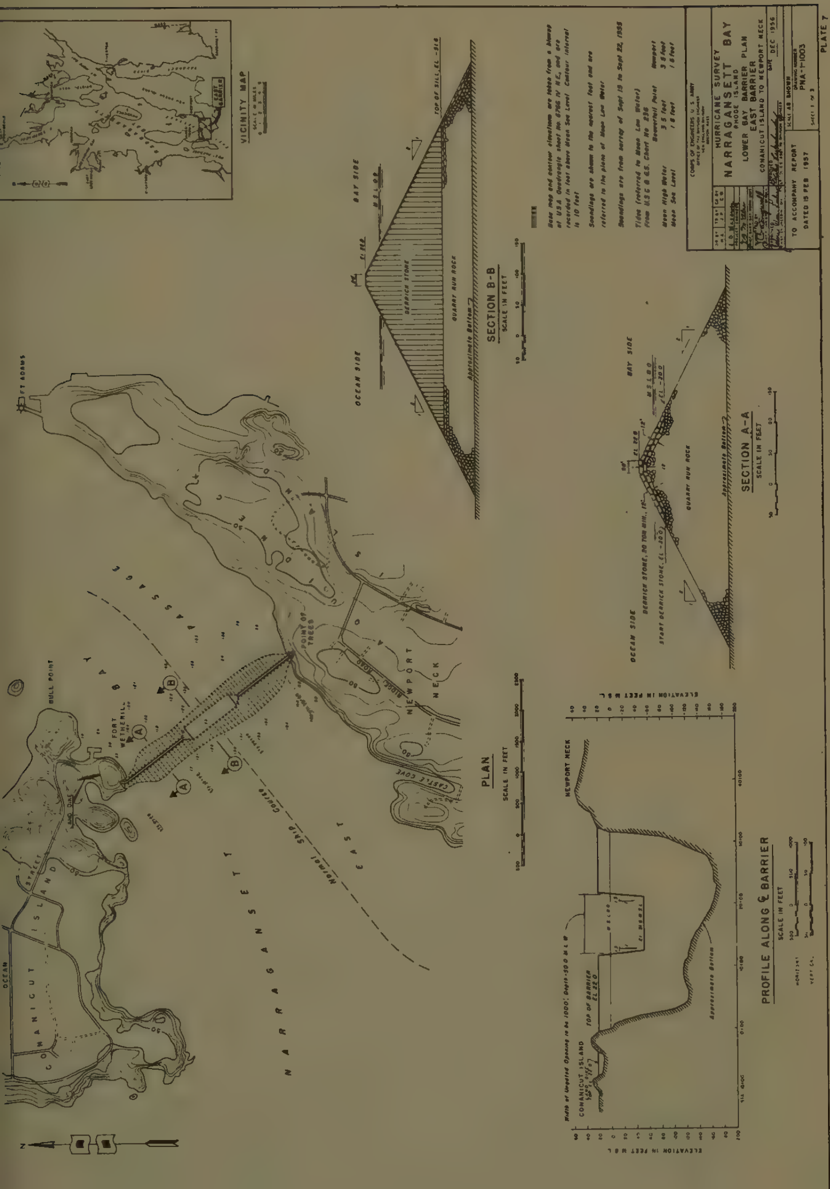
Based on studies in the Narragansett Bay and New Bedford areas, and studies in progress in coastal areas, definite methods of analyses and procedures have been adopted. It is recognized that additional basic data, studie of meteorology and tidal hydraulics of hurricane surges, and hydraulic model tests for design of structures will lead to improved procedures.

Determining Flood Damages

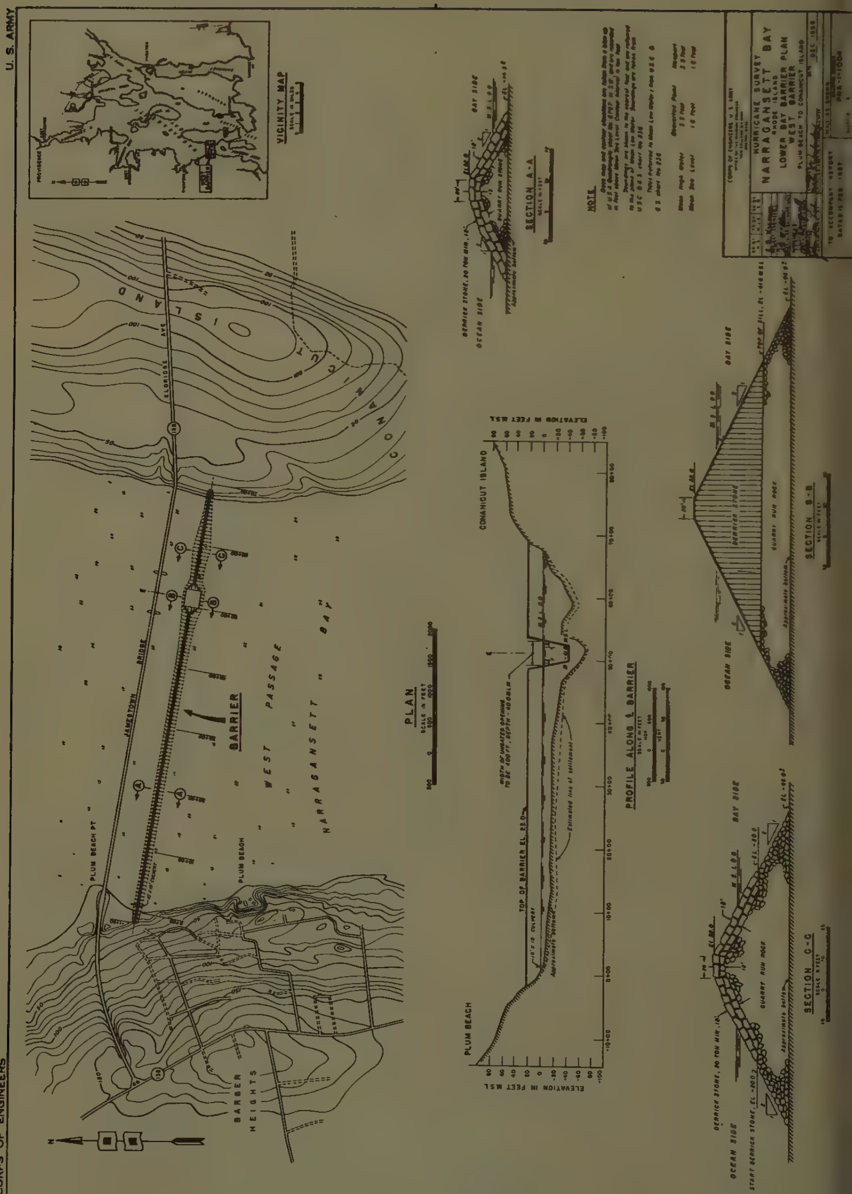
One of the first steps in hurricane protection planning is the determination of damages caused by storm-tide flooding. In New England, damage surveys were initiated in September 1955 to obtain data for economic studies of the various proposals for hurricane flood protection. Essentially the survey was a door-to-door inspection of the thousands of industrial, commercial, residential, and other properties affected by storm-tide flooding during Hurricane Carol, August 31, 1954. Data were collected on the extent and nature of the areas flooded, the depth of flooding, and the amount of damages. Loss were estimated for various stages of flooding above and below the 1954 flood level to develop stage-damage relationships such as shown on Plate 10 for th New Bedford area. Narragansett Bay and New Bedford areas sustained abou 40 percent of the $300,000,000 salt water flood damages experienced in New England during the hurricane of 1954.

Historical Study of Hurricanes

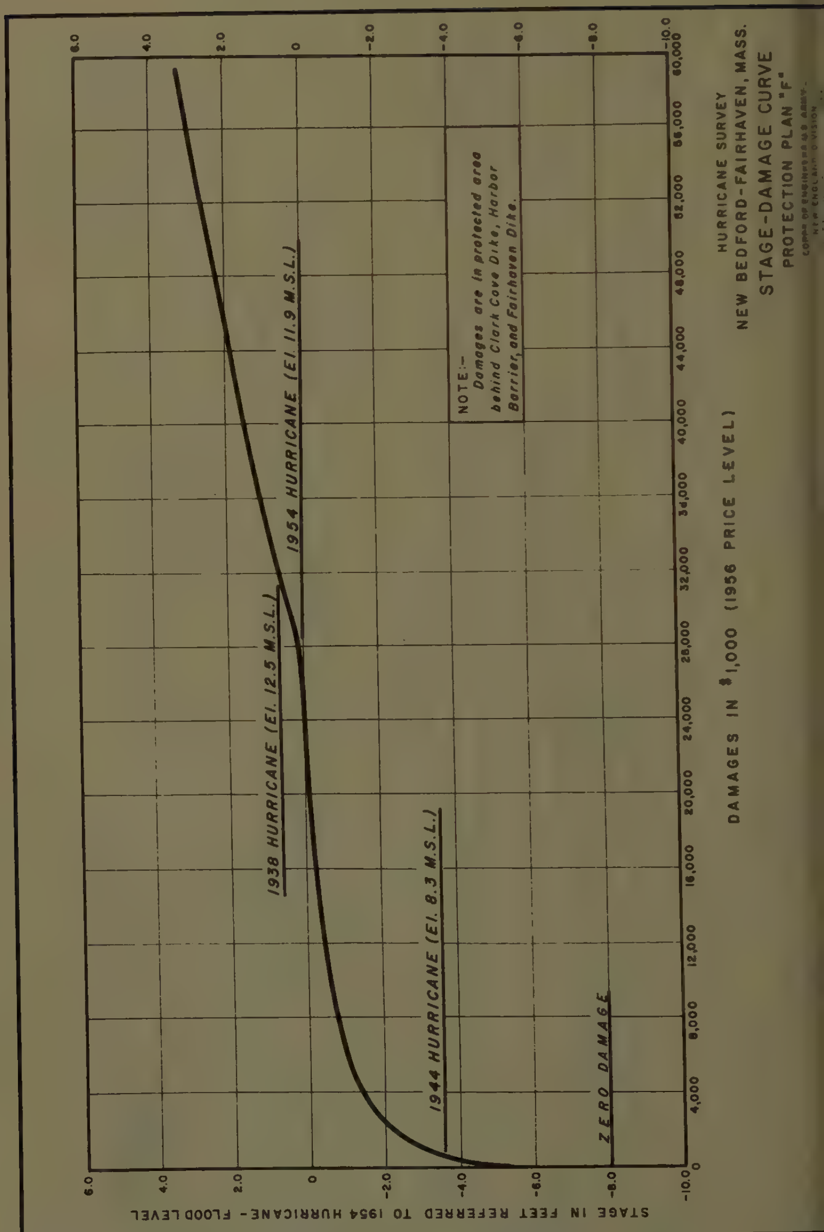
A study was made of historical and present century hurricanes, severe storms, and data relating to hurricane occurrences and high water elevation experienced. Since 1620 there have been recorded 63 damaging hurricanes c which 25 are recorded as inflicting important flood damage. Plate 11 shows flood elevations reached at Providence, Rhode Island since 1635 for 33 of the most severe hurricanes and storms for which any records of tidal elevations were available. The earliest hurricanes recorded in New England appear in Governor William Bradford's "History of Plymouth Plantations, 1620-1647," and in Governor Winthrop's "History of New England." These records describe the violent storms in 1635 and 1638 that created flood levels apparentl higher than the recent floods of 1938 and 1954. Governor Bradford wrote of the hurricane of August 1635 " . . . . It caused the sea to swell (to the

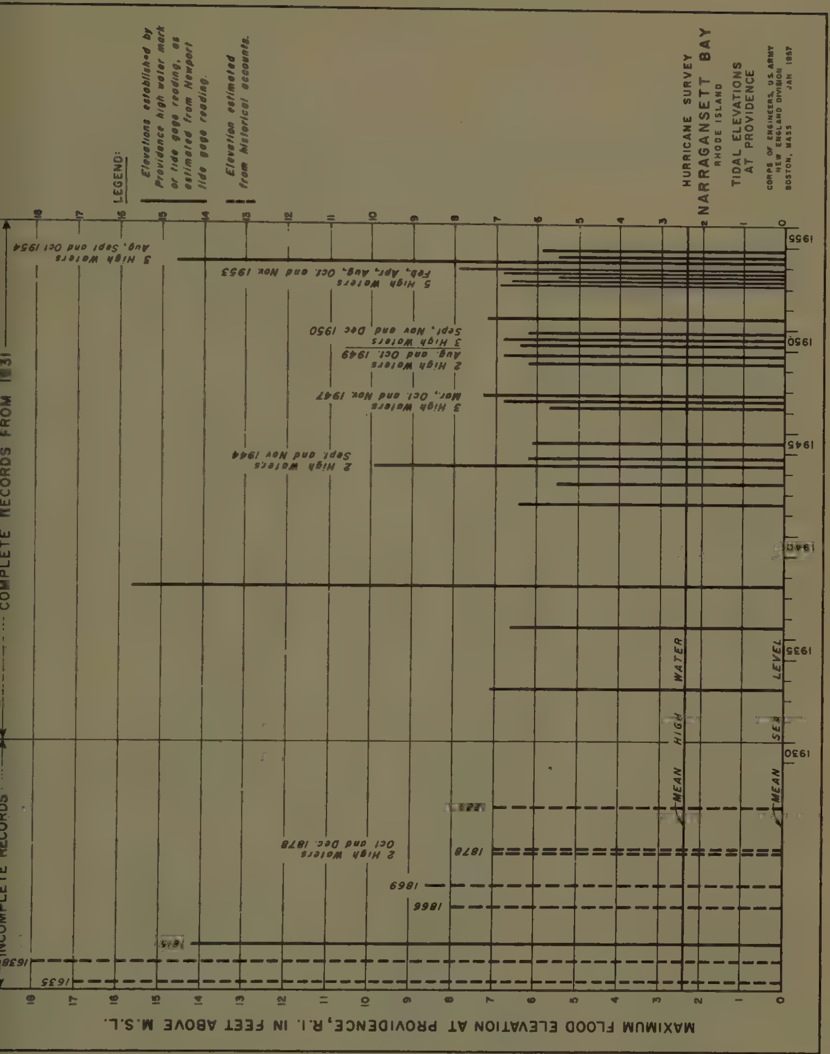


VICINITY MAP
SECTION B-B
SCALE IN FEET
OCEAN SIDE
BAY SIDE
DERRICK STONE
QUARRY RUN ROCK
Approximate Bottom
SECTION A-A
SCALE IN FEET
PLAN
SCALE IN FEET
PROFILE ALONG ℄ BARRIER
SCALE IN FEET
NEWPORT NECK
CONANICUT ISLAND
NARRAGANSETT
EAST PASSAGE
FT ADAMS
BULL POINT
FORT WETHERILL
CASTLE COVE
CORPS OF ENGINEERS U.S. ARMY
HURRICANE SURVEY
NARRAGANSETT BAY
RHODE ISLAND
LOWER BAY BARRIER PLAN
EAST BARRIER
CONANICUT ISLAND TO NEWPORT NECK
DEC 1956
PNA-1-1003
TO ACCOMPANY REPORT
DATED 15 FEB 1957
PLATE 7


U. S. ARMY
CORPS OF ENGINEERS
VICINITY MAP
PLAN
BARRIER
WEST PASSAGE
NARRAGANSETT BAY
CONANICUT ISLAND
JAMESTOWN BRIDGE
ELDRIDGE AVE
PLUM BEACH PT
PLUM BEACH
BARBER HEIGHTS
PROFILE ALONG ℄ BARRIER
TOP OF BARRIER EL. 25.0
ELEVATION IN FEET MSL
SECTION A-A
SECTION B-B
SECTION C-C
OCEAN SIDE
BAY SIDE
DERRICK STONE
QUARRY RUN ROCK
NOTE
HURRICANE SURVEY
NARRAGANSETT BAY
RHODE ISLAND
LOWER BAY BARRIER PLAN
WEST BARRIER
PLUM BEACH TO CONANICUT ISLAND


PLAN
SCALE IN FEET
BLUE BILL COVE
ISLAND PARK
PORTSMOUTH
TIVERTON
SAKONNET RIVER
BARRIER
TOP OF BARRIER EL 20.0
ELEVATION - PROFILE
SCALE HOR VER
SECTION B-B
SECTION C-C
SECTION E-E
SECTION J-J
SECTION F-F
SECTION G-G
SCALE FOR ALL SECTIONS IN FEET
CORPS OF ENGINEERS U. S. ARMY
HURRICANE SURVEY
NARRAGANSETT BAY
RHODE ISLAND
LOWER BAY BARRIER PLAN
TIVERTON BARRIER
PORTSMOUTH TO TIVERTON
DEC 1956
SCALE AS SHOWN
PNA-1-1005
TO ACCOMPANY REPORT
DATED 15 FEB 1957
PLATE 9


HURRICANE SURVEY
NEW BEDFORD-FAIRHAVEN, MASS.
STAGE-DAMAGE CURVE
PROTECTION PLAN "F"
DAMAGES IN $1,000 (1956 PRICE LEVEL)
STAGE IN FEET REFERRED TO 1954 HURRICANE - FLOOD LEVEL
1954 HURRICANE (El. 11.9 M.S.L.)
1938 HURRICANE (El. 12.5 M.S.L.)
1944 HURRICANE (El. 8.3 M.S.L.)
ZERO DAMAGE
NOTE:- Damages are in protected area behind Clark Cove Dike, Harbor Barrier, and Fairhaven Dike.


LEGEND:
Elevations established by Providence high water mark or tide gage reading, as estimated from Newport tide gage reading.
Elevation estimated from historical accounts.
HURRICANE SURVEY
NARRAGANSETT BAY
RHODE ISLAND
TIDAL ELEVATIONS AT PROVIDENCE
CORPS OF ENGINEERS, U.S. ARMY
NEW ENGLAND DIVISION
BOSTON, MASS JAN 1957
PLATE 11
MAXIMUM FLOOD ELEVATION AT PROVIDENCE, R.I. IN FEET ABOVE M.S.L.
INCOMPLETE RECORDS
COMPLETE RECORDS FROM 1931
MEAN HIGH WATER
MEAN SEA LEVEL
3 High Waters Aug, Sept and Oct 1954
5 High Waters Feb, Apr, Aug, Oct and Nov 1953
3 High Waters Sept, Nov and Dec 1950
2 High Waters Aug. and Oct. 1949
3 High Waters Mar., Oct. and Nov. 1947
2 High Waters Sept and Nov 1944
2 High Waters Oct and Dec. 1878
1878
1869
1866
1815
1638
1635
1955
1950
1945
1940
1935
1930

southward of this place) above 20 feet, right up and down, and made many o the Indians to climb into trees for their safety:" . . . . Governor Winthrop wrote: "The tide rose at Narragansett 14 feet higher than ordinary and drowned 8 Indians flying from their wigwams." Of the storm of August 163 Governor Winthrop wrote: "It flowed twice in 6 hours and about Narragans it raised the tide 14 or 15 feet above the ordinary spring tides, upright."

Very early accounts of hurricanes in the area are brief, but since 1815, and particularly the period from 1901 to the present, accounts have become increasingly more complete and scientific. This is due to the increase in trained observers and rapid advances in the knowledge of meteorological phenomena. . . . .

Data on Recent Hurricanes

The three most damaging hurricanes since 1900 occurred in the 17-year period between 1938 and 1954. Two of these, the hurricane of September 19 and August 1954 (Carol), produced flood levels of about 15 to 16 feet above mean sea level at Providence. The upper portion of Plate 12 shows the 193 hurricane flood levels in Narragansett Bay, both the still-water elevation p file and the profile including wave action, as developed after extensive field surveys. Plate 13 shows the hurricane tide graphs for the recent hurricane at New Bedford and their relation to gravitational tide. The tidal surge, or storm surge (the maximum height of hurricane tide above astronomical tide is a maximum of about 10 feet in the 1938, 1944, and 1954 hurricanes, Both the 1938 and 1954 hurricanes were nearly coincident with astronomical high tide which contributed significantly to the severity of the flooding. The hur cane of September 1944, although a severe storm, struck at a time of low ti and flooding was consequently less severe.

Determination of Design Flood Levels

Although definitive studies have not been completed of the maximum sto and flood-surge possibilities in the New England area, a design storm was developed by a combination of analytical methods and hydraulic model studi to demonstrate the potentialities of tidal flooding.

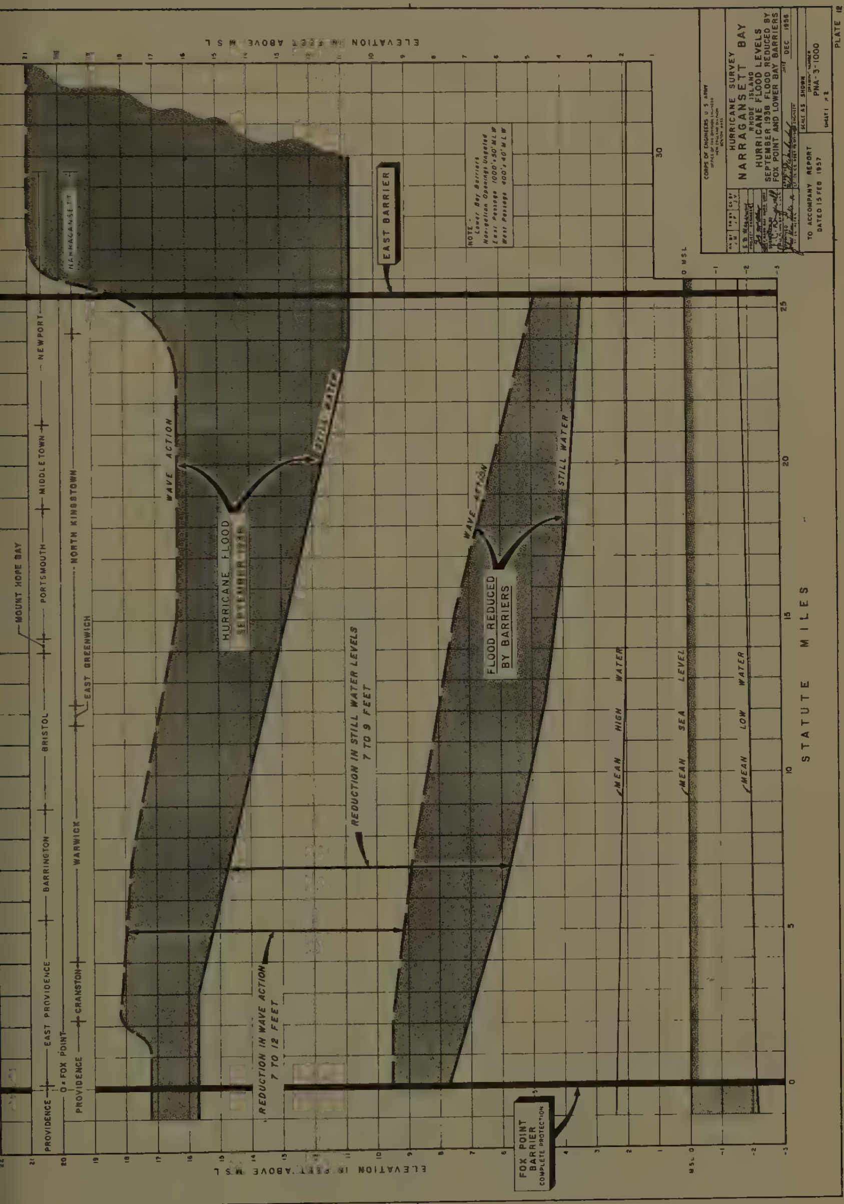
The work in the Narragansett Bay area involved the following steps:

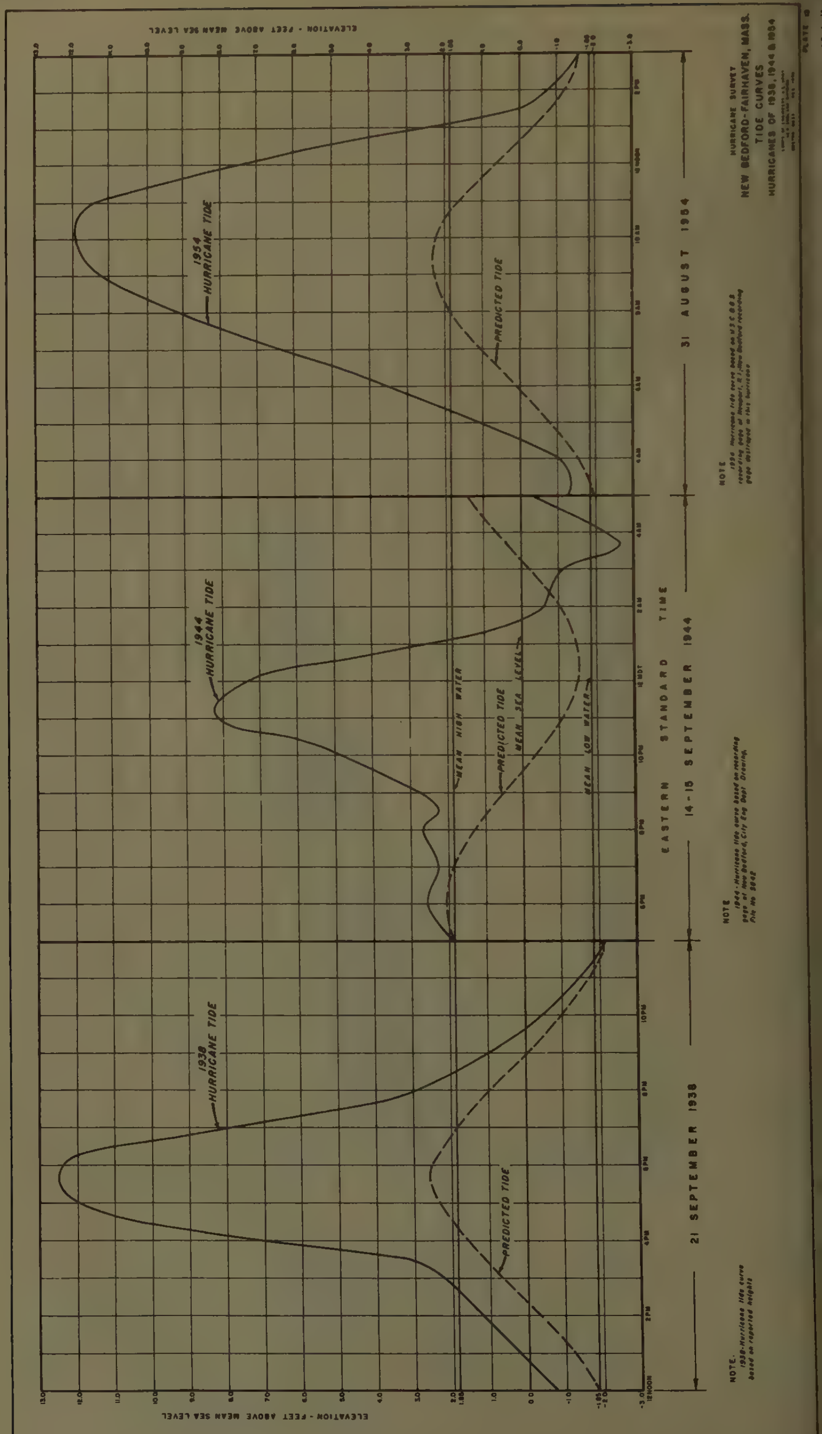
1. Determination of design hurricane with associated wind direction and velocities and barometric pressures, based on a transposition of the Septer ber 1944 hurricane. Storm track and forward speed were adjusted to produ most critical conditions.

2. Calculation of storm surge at the coast (mouth of Narragansett Bay) the method of Dr. R. O. Reid.(1) Recorded flood elevations were used to c these calculations. The design surge, above astronomical tide, was estima to be 10.3 feet for a storm speed of 20 knots and 9.5 feet for a speed of 40 knots.

3. Routing of the storm surge up Narragansett Bay was accomplished b hydraulic model tests, described later, and checked by analytical methods. The studies indicated an increase in the height of the surge within the bay varying from 2 to 3 feet depending on the storm speed.

4. Wind set-up in the 25.5 mile fetch of the bay could not be simulated i the hydraulic model and was calculated by the methods of Dr. R. O. Reid(2)


CORPS OF ENGINEERS U.S. ARMY
HURRICANE SURVEY
NARRAGANSETT BAY
RHODE ISLAND
HURRICANE FLOOD LEVELS
SEPTEMBER 1938 FLOOD REDUCED BY
FOX POINT AND LOWER BAY BARRIERS
DEC 1956
SCALE AS SHOWN
PNA-3-1000
TO ACCOMPANY REPORT
DATED 15 FEB 1957
PLATE 12
NOTE - Lower Bay Barriers
Navigation Openings Ungated
East Passage 1000'x50'MLW
West Passage 400'x40'MLW
EAST BARRIER
FOX POINT BARRIER COMPLETE PROTECTION
HURRICANE FLOOD SEPTEMBER 1938
FLOOD REDUCED BY BARRIERS
WAVE ACTION
STILL WATER
REDUCTION IN STILL WATER LEVELS 7 TO 9 FEET
REDUCTION IN WAVE ACTION 7 TO 12 FEET
MEAN HIGH WATER
MEAN SEA LEVEL
MEAN LOW WATER
ELEVATION IN FEET ABOVE M.S.L
STATUTE MILES
PROVIDENCE
0 = FOX POINT
EAST PROVIDENCE
CRANSTON
BARRINGTON
WARWICK
BRISTOL
EAST GREENWICH
MOUNT HOPE BAY
PORTSMOUTH
NORTH KINGSTOWN
MIDDLE TOWN
NEWPORT
NARRAGANSETT


HURRICANE SURVEY
NEW BEDFORD-FAIRHAVEN, MASS.
TIDE CURVES
HURRICANES OF 1938, 1944 & 1954
ELEVATION - FEET ABOVE MEAN SEA LEVEL
1954 HURRICANE TIDE
1944 HURRICANE TIDE
1938 HURRICANE TIDE
PREDICTED TIDE
MEAN HIGH WATER
MEAN SEA LEVEL
MEAN LOW WATER
EASTERN STANDARD TIME
21 SEPTEMBER 1938
14-15 SEPTEMBER 1944
31 AUGUST 1954
NOTE
1938-Hurricane tide curve based on reported heights
NOTE
1944-Hurricane tide curve based on recording gage at New Bedford, City Eng Dept Drawing, File No 3042
NOTE
1954 Hurricane tide curve based on U.S.C.&G.S. recording gage at Newport, R.I. New Bedford recording gage destroyed in this hurricane

and the approximate methods discussed later. Calculations using a steady wind of 76 miles per hour gave a wind set-up of approximately 3 feet. Studies indicated that the transient winds of a hurricane might increase the wind effect to 5 feet.

5. The total of design surge plus wind set-up on top of a spring tide was calculated to be 18.7 feet m.s.l. at Providence, and 13.0 feet m.s.l. at Newport.

An approximate method of deriving design storm-tide flood level, based on the Narragansett Bay studies, was used for the New Bedford project. The 1938 hurricane storm tide potential off Newport, Rhode Island, at the entrance to Narragansett Bay, has been calculated to be 1.25 times as great as the 1938 surge. Using this information, in lieu of analytical determinations by the methods of Dr. R. O. Reid, a design storm surge was computed for New Bedford and Fairhaven Harbor, as follows:

1938 HURRICANE FLOOD DATA

Observed Peak flood level, 1938 hurricane	12.5	feet msl
Predicted tide for time of 1938 peak	2.5	feet msl
Storm surge	10	feet
Calculated 1938 wind set-up	2	feet
Estimated Storm surge less wind set-up	8	feet

ESTIMATED DESIGN FLOOD DATA

Design storm surge, without wind set-up, 25% greater than 1938 - 8x1.25 =	10	feet
Calculated Design storm wind set-up	5	feet
Design storm surge	15	feet
Predicted Spring tide	3	feet msl
Estimated Design flood still water level	18	feet msl

Under design hurricane conditions for New Bedford and Fairhaven Harbor, the storm-tide potential, estimated at 15 feet, superimposed on a spring tide gives a storm-tide flood level of 18 feet msl, or 5.5 feet higher than the level experineced in 1938.

Wind set-up calculations as in the above tabulation are determined from either of two formulas:

a) For a problem requiring determination of the profile of the tilted water surface created by the wind on the fetch between the windward and the leeward sides of the body of water a step method formula (1) is applicable as follows:

$$\Delta S = d_T \left[\sqrt{\frac{1 + 2KU^2 \Delta F}{d_T^2}} - 1 \right]$$

where

ΔS = incremental wind set-up in feet
d_T = average depth of water in feet

U = velocity of wind in miles per hour
ΔF = incremental fetch length in statute miles
K = coefficient (1.165×10^{-3})

b) Where the chief concern is determination of the effect of wind set-up on the leeward side of the body of water, as at New Bedford, this can be approximated in a single calculation by use of the formula (4):

$$S = 1.165 \times 10^{-3} \frac{V^2 F}{D} \cos A$$

where S = the total difference in wind tide between windward and leeward shores.
V = wind velocity in miles an hour
F = fetch in statute miles
D = average water depth in feet
A = angle between the wind and tidal axis

With the above formula and using maximum winds of 56 mph that were experienced in 1938 and 88 mph in the design storm, wind set-up of 2 and 5 feet, respectively, are determined for the 12-mile fetch between the Elizabeth Islands and New Bedford. These calculations give the rise in an open body of water as the surge moves up a bay or estuary.

In Narragansett Bay the wind set-up problem, without hurricane protection barriers, is similar to New Bedford's. However, the bay problem is greatly complicated by the irregularities of configuration and hydrography in the 25.5 mile fetch and the wind pattern that might apply between the mouth of the bay and Providence.

If Narragansett Bay is partially closed off from the ocean by hurricane barriers, the wind set-up would cause an actual transfer of water from the windward to the leeward end of the bay. The amount of the depression at the windward end of the bay will depend on:

a) the area of the opening in the barrier which supplies water from the hurricane surge in the ocean

b) time lag of the hurricane surge with respect to maximum wind velocities which cause wind set-up within the bay, and

c) the shape of the bay and distribution of its water areas and depths.

Regardless of the depression at the windward end, the wind set-up formula gives a good approximation of the tilting of the water surface for any location. In an inland lake of fairly regular shape and depth, 40 percent of the total wind set-up would occur as depression and 60 percent as an increase above the normal level.[5]

Calculation of Design Waves and Wave Run-up

Wave heights and wave run-up were necessary to assist in setting the top grade of considered structures and as a feature in structural design. Wave heights can be determined approximately from curves showing the relationship of (1) fetch distance, (2) wind velocity, (3) wind duration, and (4) wave height and period, or more precisely, with depths, shoaling and frictional

efficients in incremental fetches along with the formulas:

(1) $$H_o = .0555\sqrt{V^2 F}$$

(2) $$T_s = .5\sqrt[4]{V^2 F}$$

where

H_o = significant deep water wave height, feet

T_s = significant deep water wave period, seconds

V = wind speed, knots (constant)

F = fetch length, nautical miles

l of which is presented by Bretschneider.(6)

stimating Design Rainfall and Run-off

The next step was to determine the rainfall and run-off which is associated ith hurricanes. Heavy rainfall is normally associated with great hurricanes. arm, moisture-laden air is carried in over the land from the ocean by winds advance of a hurricane causing heavy precipitation before and at the time e hurricane strikes. Consequently, high river run-off and severe flooding e likely to occur at the same time as a hurricane tidal-flood. An example this situation was the hurricane flood of September 1938 when many of the vers in New England overflowed their banks and caused extreme flood sses one day in advance of the hurricane, while on other rivers, such as the hames River above New London, Connecticut, fresh water flooding occurred o hours before salt-water flooding.

The September 1938 storm which had the greatest rainfall of record sociated with hurricane winds and the highest recorded tide levels, was opted for design studies in connection with Narragansett Bay. The storm infall, with its center near Middletown, Connecticut, where a total of 17 ches was recorded for the period September 17-21, was transposed to the ea above the proposed tidal barriers.

For studies of the area above Fox Point in Providence, with a drainage ea of 77 square miles, the 24-hour storm rainfall of 9.5 inches was centered er the area to produce the maximum amount of run-off at this point. An inw hydrograph for the transposed 1938 storm at Fox Point was developed om unit hydrographs. Rainfall distribution applied to the unit graphs was sed on graphs of "Maximum Average Depth-Duration of Rainfall"(8) for New gland over a drainage area of 100 square miles. It was determined that the sign fresh water hydrograph at the Fox Point location would have a peak scharge of 9,200 c.f.s. and a total volume of about 21,000 acre-feet, or the uivalent of 5.1 inches of run-off from the 77 square miles of drainage area. e storage available above the barrier is less than 950 acre-feet at elevan 6.5 m.s.l. (approximate elevation of zero damage in Providence), so that rge pumps are required to pass the design run-off while the gates are osed.

Determining Overtopping

The volume of water that would overtop protective barriers is important not only for the design of a safe structure, but from the standpoint of prevention of flooding and the design of an adequate drainage and pumping system to remove the overtopping water. The Fox Point barrier, consisting of a concrete gravity section 40 feet in width with a deck elevation 12.5 feet m.s.l. and a concrete wall 10 feet higher extending the full length of the barrier on the upstream side, was determined by hydraulic model tests to be particularly effective in minimizing overtopping. Under design hurricane conditions with still water elevation 18.7 feet m.s.l. and significant design waves of 6 feet with a period of 5 seconds, the overtopping rate averages about 0.2 c.f.s. per foot for the 900-foot effective length. This is equivalent to approximately one-third of a foot rise over the 40 acres of water area during the hour it is estimated overtopping would take place. With the maximum depth of water on the deck limited to 6.2 feet, the maximum wave height that is supported is .78 x 6.2 feet, or about 5 feet.* Wave heights in the wave spectrum** greater than those that will be supported by the water depth are broken before reaching the vertical wall and little or no overtopping occurs. The basic reference for the overtopping calculation was Technical Memorandum No. 64 of the Beach Erosion Board.(9)

Design of Protective Structures

The locations of the New Bedford and Narragansett Bay barriers were governed by existing developments and by considerations of necessary length, water depth and foundation conditions. Structures were planned wherever possible to take advantage of topographic features for economy and efficiency. Especially where large natural forces such as hurricane storms are involved, the severity and extent of these forces may be significantly altered and modified by underwater features and ground forms. Good information on the physical setting of proposed structures is essential. The main steps in the design of the structures are outlined below.

1. Topographic and Hydrographic Surveys. Existing topographic maps and hydrographic charts were used as base maps where possible, corrected to include recent development, and amplified by detailed field surveys of dam sites. Aerial photographs were also used.

2. Foundation Explorations. Particularly for marine structures, foundation conditions are likely to be a controlling element in determining settlement, stability and the economic feasibility of the structures. Preliminary investigations included collection of available data on explorations in the vicinity and studies of the general geology of the area. In Lower Narragansett Bay bottom sampling was accomplished by means of a Kulenburg Corer, giving 6 to 10 feet of penetration. In addition, seismic methods were employed. These methods mostly indicated the need for core borings which will be very costly in the 165 foot deep water of the East Passage of Lower Narragansett Bay. At most of the sites investigated, however, borings were obtained for both water crossings and land dikes.

* Height of breaking wave, H_b = .78 depth of water.

** Wave height frequency distribution after Putz, & Longuet-Higgins.

3. Source of construction materials. The structures proposed are exten- e enough to require large amounts of construction materials, such as regate and stone. Preliminary studies were made of the available sources naterials from the viewpoint of accessibility and cost. For construction he Lower Bay barriers, at least 7,000,000 cubic yards of rock would be uired, including both quarry-run and heavy derrick stone for capping. large quantities would require opening new quarries near the sites, and structing a fleet of bottom-dump barges for the placing of the stone.

4. Slopes of Rockfill Barriers. Side slopes vary from 1 on 1 1/2 to 1 on 2 ending on the exposure to wave action. This is a compromise between re- rements of stability and economy. Flatter slopes are required for poor ndation conditions.

5. Stone Sizes. For wave resistant structures such as the East Barrier, tected on the top and sides by armor stone, the sizes of stones and the kness of the layers becomes a major design factor. Successive layers of ious weight stone are required to protect against the tremendous dynamic ces of breaking hurricane waves. The weight and thickness of layers varies n a maximum for the outer layer to a minimum for the inner layer adja- t to the quarry-run rock of the interior of the structure. The method used computing the weight and thickness of the layers of stone was obtained n Technical Report No. 4 of the Beach Erosion Board,(10) and from unpub- ed papers of Mr. Robert Y. Hudson of the Waterways Experiment Station, ksburg, Mississippi,(11) which summarizes recent research on rubble nd structures.

The general formula for weights of armor units, as developed by Mr. lson, is as follows:

$$W_r = \frac{\gamma_r H^3}{k_D (S_r - 1)^3 \cot \alpha}$$

where

W_r = Weight of armor unit, lb.

γ_r = Specific weight, lb./ft.^{3} (170 lb. approximately)

H = Wave height, ft.

k_D = Damage coefficient

α = Angle of breakwater slope, measured from the horizontal, degrees

S_r = Specific gravity of armor unit relative to water in which the breakwater is located. (2.65 approximately)

above equation allows for displacement of a percentage of the armor e, based on the selected k_D factor, as shown in the table below:

Displacement of Armor Units in Percent of Total	k_D
30 - 60	15.9
15 - 40	12.8
10 - 20	9.5
5 - 15	7.2
1 - 5	5.1
0 - 1	3.2

The selection of the proper k_D factor would depend upon considerations such as the probable duration and frequency of wave action; practical sizes of armor units and the cost of maintenance; the possibility of a second hurricane occurring before the damage caused by the first could be repaired; and the probability of loss of life if failure occurred. The armor units are place in a minimum of two layers, of which the combined thickness may be found b the following formula where t = thickness in feet:

$$t = 2 \left(\frac{W_r}{\gamma_r} \right)^{1/3}$$

Immediately under the armor stone, layers of bedding stones are required.

For the stones of twenty ton minimum weight proposed for the structures in Lower Narragansett Bay, there would be some damage in the period of on to two hours when design waves would be breaking near the top of the structure. This would be reflected in higher maintenance costs, and probably would require keeping a quarry open on a standby basis. But stones of such size as to be theoretically completely stable against the highest design wave would be extremely expensive to quarry, transport, and place. A thorough study of stone sizes, based on actual stone sizes available in the quarry, would be required on the final design of the project.

Shapes other than massive stone blocks have been used in some coastal engineering projects. Where stone is not locally available in sufficient size, special prefabricated concrete shapes as tetrahedrons or tetrapods[12] may be used. With a relatively high initial cost of material, the prefabricated shapes may prove more economical in some cases. The prefabricated shap can be of smaller size than the stone blocks and will provide the same stability as the stone blocks, which results in smaller placing costs. In the final design of the Lower Bay barriers, special shapes may be used for greater efficiency in dissipating wave energy.

6. Forces on Structures Due to Waves. Protective structures on expose shores are generally located in depths of water that would cause breaking o the greatest hurricane waves seaward of the structure. Where a masonry structure or navigation gates are located in relatively deep water, non-breaking wave conditions may occur. The method used for the determinatio of pressure due to these waves is that of Sainflou.[10] Most protective stru tures are located in depths of water where some waves will break directly c the structure. The combination of dynamic and hydro-static forces develop by breaking waves was developed by the Minikin Method.[10]

Effect of Structures on Storm-Tide Flooding

The effectiveness of preventive measures was apparent as alternative
is were studied. The limitations of any method of protection against the
ft onslaught of hurricane flood and giant storm waves is an important
sideration.

The studies of various plans of dikes, flood walls and barriers were made,
ping in mind the principal factors that control the degree of protection,
nely, (1) top elevation as it controls overtopping by flood and waves; (2) in-
of salt water through ungated openings; (3) inflow of fresh water from
l storm run-off and stream flow; (4) the rise in pool level (resulting from
(2) and (3); (5) local wind and wave effects within the pool; and (6) build-
n levels below barriers. As discussed below, these factors were deter-
ed by a combination of analytical methods and hydraulic model tests.

l. Top elevation. - A high degree of protection against overtopping is
ally considered necessary in densely populated areas, but it may not be
ctical to design a structure high enough to prevent all wave overtopping.
stantial quantities of water may be carried over a dike or wall along with
d, gravel, and large boulders, and cause heavy damages. Overtopping of a
e or flood wall may result in more damage than would occur without the
icture and also cause destruction of the protective structure itself.

2. Inflow of salt water through openings. - Gated openings such as the
tor gates for navigation at New Bedford prevent inflow of salt water from
offshore hurricane surge. However, the preliminary design of the Lower
barriers in Narragansett Bay is for ungated structures except for a small
igation gate at Tiverton. The key to effective design of such barriers is:

a) Making the opening small enough to effectively restrict the entrance of
ricane surges and provide a reasonable degree of protection in the bay as
hole, and

) Providing openings of large enough width and depth for navigation with-
producing excessively fast currents under normal conditions.

3. Inflow of fresh water. - With determination of local storm run-off and
eam flow from the design rainfall during the period of a hurricane surge,
eral alternative methods for the disposal of fresh water may be con-
ered.

) Pumping over the dike or barrier.

) Pressure conduits beginning outside of the flood area and passing
ugh the structure.

) Storage or natural ponding within the protective area, and

) Reservoir storage above the flood area.

) Diversion of storm sewers and river flow past the dike or barrier or
an adjacent watershed.

. Rise in pool level. - The combination of salt water inflow through un-
d openings, overtopping by waves and fresh water run-off all tend to pond
lood the protected area. It is necessary to determine the relation between
d water level within the protected area and level of basements, streets and

other locations where flooding begins in order to prevent appreciable damag from this rise in water level. For the New Bedford project, with gated open ings to prevent salt water inflow, the area of the harbor was sufficient to ab sorb the fresh water inflow without damage. In Narragansett Bay, pumps ar provided for the Fox Point project at the head of the bay, while the water ar of 120 square miles back of the Lower Bay Barriers is sufficient to absorb considerable inflow and runoff before the bay rises to a damaging flood leve

5. Local wind and wave effects within the pool. - An off-shore barrier located many miles from a damage center will not provide complete protection because:

a) Wind set-up will cause an upward tilting of the water surface within th protected pool area, as described on Page 21 and 22.

b) Waves will be generated in the fetch between barrier and damage cent

6. Build-up in levels below barriers. - A barrier blocking the tidal surge causes an increase in flood levels downstream. Model tests show this build up to be as high as two or three feet for barriers located in the middle of Narragansett Bay where build-up is a maximum. The build-up was found to be small below barriers or dikes located at the head of the bay or near the mouth of the bay because of the large proportion of water area below the bar riers compared with the relatively small area behind the barriers. These tests showed where not to build barriers; that in some locations barriers wi cause an increase in flood levels.

Effect of Barriers on Normal Oceanographic Conditions

Barriers crossing rivers, estuaries or bays are likely to have important effects on oceanographic conditions. In Narragansett Bay, for example, the effects of the Lower Bay barriers on the normal conditions of tides, curren temperature, salinity, flushing, sedimentation, fisheries, pollution and navig tion, are exceedingly complex and require extensive investigation over a period of years. As a result of these studies larger navigation openings in barriers may be required and sluice gates added which would greatly dimin the effect the barriers have in restricting tidal circulation.

Preliminary tests of the effects of the tidal barriers on oceanographic co ditions were made in the hydraulic model of Narragansett Bay after the mod was adjusted to reproduce existing tides and currents. Basic information o present conditions in the bay was derived from the considerable data on tem peratures, salinity, flushing rates and silting in the bay collected during 195 by the Narragansett Marine Laboratory of the University of Rhode Island, a from observations of the U. S. Coast and Geodetic Survey.

Hydraulic Model of Narragansett Bay

One of the most valuable tools in the complex engineering studies of hur cane protection in Narragansett Bay is a hydraulic model which has been co structed at the Waterways Experiment Station of the Corps of Engineers at Vicksburg, Mississippi.

ription of the Model

e model (See Plate 14) at the Waterways Experiment Station reproduces Narragansett Bay and a portion of the Atlantic Ocean adjacent to the bay nce and consists of a fixed-bed construction with scale ratios, model to type, of 1:1000 horizontally and 1:100 vertically. The model is approxi-ly 100 feet wide and 250 feet long.

The first use of the model was to reproduce the normal ocean tides and nts using fresh water to approximate conditions in the bay. The normal nomical tides are produced in the model by a pumping system controlled fferent cams which represent mean tide, spring tide, or the predicted which occurred with the 1938 hurricane surge. At a large number of tide and measuring points the Bay and the model are checked against each using information from extensive hydrographic surveys. Detailed hy-raphic surveys form a basis for verification of the model with the proto- With barriers in place the model shows the effect of barriers on normal tides.

The second main use of the model was to test under hurricane conditions with and without barriers. A surge machine drives a tidal surge into the l in much the same way the wind-driven surges of 1938 and 1954 rushed he bay. The surge machine is a 30-inch steel beam, 28 feet long, running cks. Without barriers the model reproduced the hurricane surges as occurred in Narragansett Bay, that is high water elevations measured in ay and in the model check quite closely.
is important to note that the model does not reproduce the local wind p and waves. These factors were computed and added to the test results hydraulic model.

The third use of the model is for detailed studies of the effect of bar- on normal oceanographic conditions in Narragansett Bay, using water ted to the proper salinity. Tests are now in progress involving mixing, ing, and sedimentation. These factors will give an indication of the effect barriers on currents, salinity, temperature, water quality and marine the bay.

Analytical Routings

advance of hydraulic model studies of Narragansett Bay, analytical rout-alculations were made for numerous barrier locations with variations in nsions of navigation openings. These data supplemented studies at the rways Experiment Station. Calculations of velocities and water surface tions were based on the routings predicated on storage in the 120 square water area above the barriers and the basic formula:

$$Q = CA\sqrt{2\ gh}$$

e

Q = rate of discharge, for a short period of time corresponding to "h" during the period, cubic feet per second

C = coefficient of discharge, varying between 0.6 and 0.8, depending on the relative size and shape of the openings and type of construction

A = average cross-sectional area of opening, square feet

g = acceleration of gravity, 32.2 feet per second per second

h = difference in water surface elevation between ocean tide and Bay level, feet

'he formula does not evaluate all the variable losses from contraction, ex-sion, friction, wind, and other indeterminate factors, but provides reason-results, checking quite closely with model investigations for velocities ugh openings and effect on tidal fluctuations as to elevations and timing.

Economics of Hurricane Protective Works

'he benefit-cost determinations for hurricane planning followed the hods of analysis which are in general use for river flood control works. first step was to determine average annual flood losses and flood preven-benefits, as follows:

ermining Annual Losses and Benefits

. Flood frequency determination. - Flood elevations were determined n recent records and estimated from historical studies to make full use of limited data available. The tidal flood elevations were then arranged in er of magnitude and plotting positions calculated in percent chance of oc-ence in any one year. The elevation-frequency curve is given in Plates nd 16 for New Bedford and Narragansett Bay, respectively.

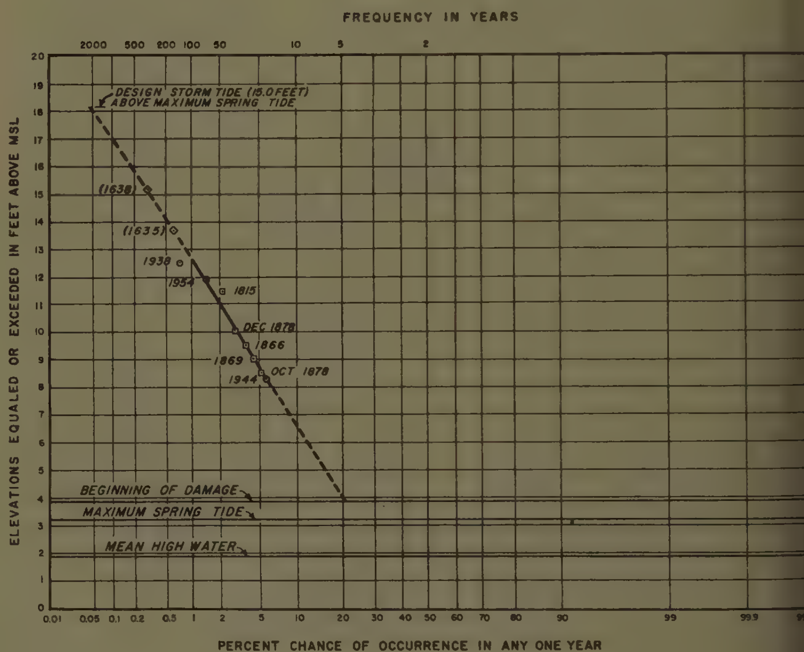
. Stage-damage curve. - The estimated damages from tidal flooding were elated with flood stage (elevation) and plotted. See Plate 10 for New Bed-.

. Damage-Frequency Curve. - By combining the flood elevation vs fre-cy and the flood elevation vs damage relationship a graph of flood damage requency was prepared. Plate 17, of the damage-frequency curve for the Bedford-Fairhaven area, shows the curve for the "Natural" or existing ition and the "Modified" condition as protected by the hurricane barrier.

. Average annual flood losses and benefits. - The average annual losses New Bedford-Fairhaven from the area under the "Natural" curve, Plate 17, determined at $949,200. Residual damages from ponding behind dike, topping and run-off in great storms was determined from the area under "Modified" curve as $5,400. The average annual benefits from flood ages prevented equals the difference or $943,800. The New Bedford pro-was also credited with additional benefits for savings in temporary protec-measures and emergency precautions set into operation upon threat of a icane.

rmining Costs and Annual Charges

stimates of first costs were prepared using unit prices based on actual rices for similar work and allowing for the cost of lands, easements, and ts-of-way, modification of sewers and utilities, and related costs. Annual ges were based on 2.5 percent interest on the investment amortized over -year period, plus the estimated costs of maintenance and operation.



LEGEND

⊙ ELEVATION FROM SURVEY OF HIGH-WATER MARKS

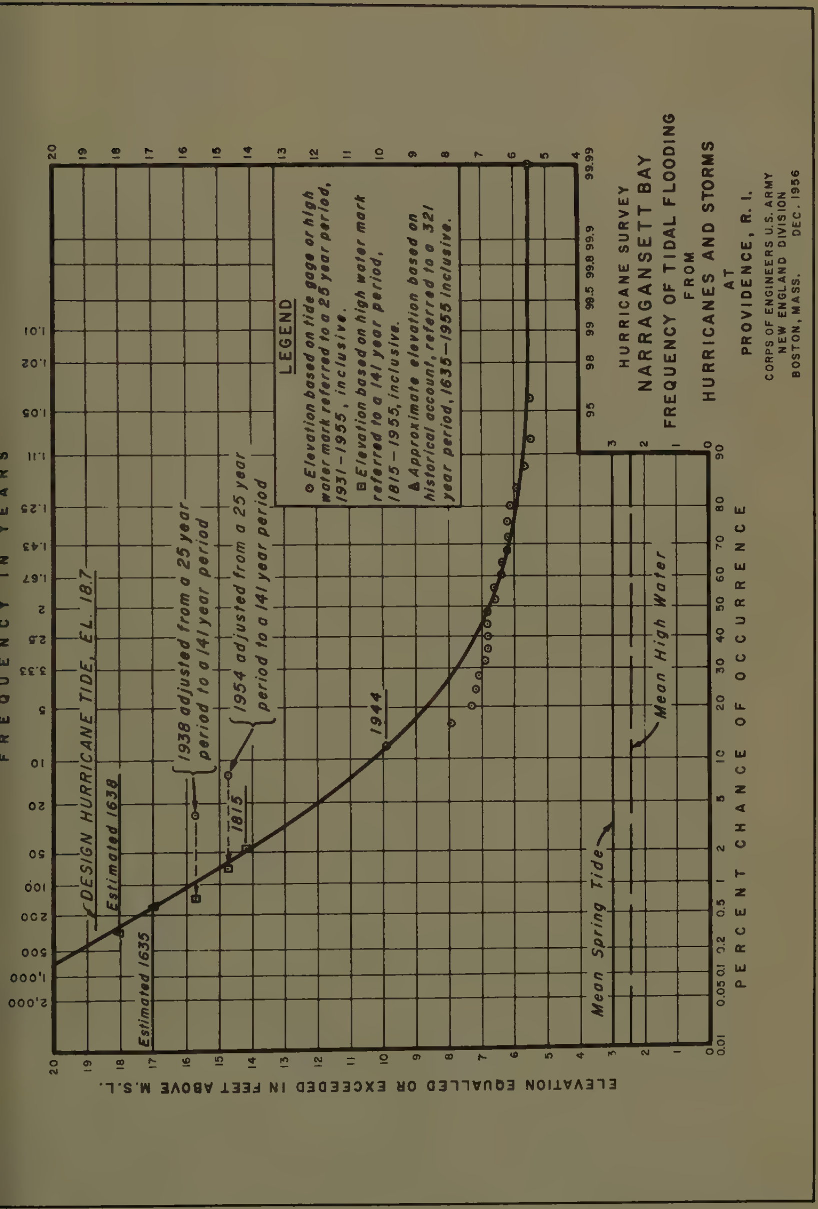
⊡ ELEVATION ESTIMATED FROM HISTORICAL DESCRIPTIONS OF GENERAL NATURE

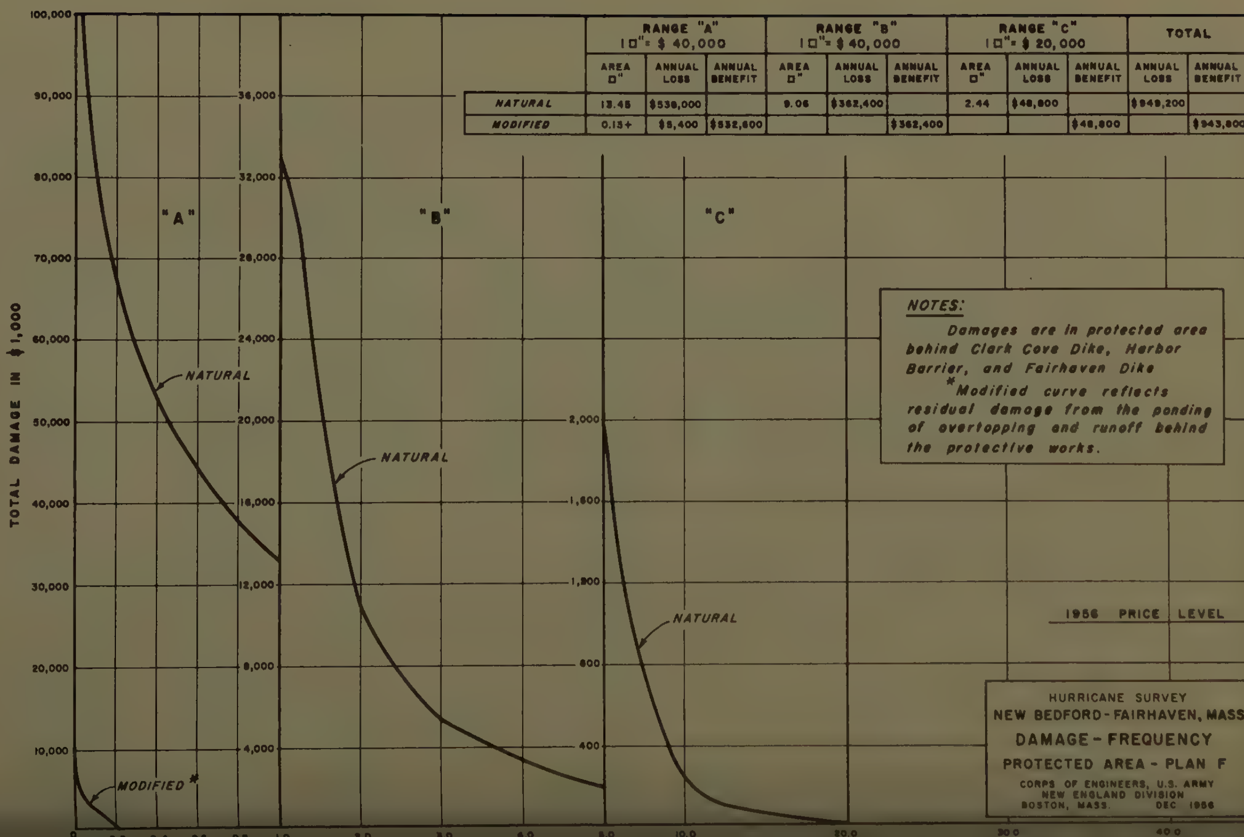
◇ FREQUENCY BASED ON PERIOD 1635-1955

HURRICANE SURVEY
NEW BEDFORD-FAIRHAVEN, MAS
ELEVATION-FREQUENCY CURVE
DAMAGING HURRICANES
1815 – 1955

CORPS OF ENGINEERS U S ARMY
NEW ENGLAND DIVISION
BOSTON, MASS. DEC. 1956

PLAT


FREQUENCY IN YEARS
ELEVATION EQUALLED OR EXCEEDED IN FEET ABOVE M.S.L.
PERCENT CHANCE OF OCCURRENCE
DESIGN HURRICANE TIDE, EL. 18.7
Estimated 1638
Estimated 1635
1938 adjusted from a 25 year period to a 141 year period
1954 adjusted from a 25 year period to a 141 year period
1815
1944
Mean Spring Tide
Mean High Water
LEGEND
⊙ Elevation based on tide gage or high water mark referred to a 25 year period, 1931-1955, inclusive.
⊡ Elevation based on high water mark referred to a 141 year period, 1815-1955, inclusive.
▲ Approximate elevation based on historical account, referred to a 321 year period, 1635-1955 inclusive.
HURRICANE SURVEY
NARRAGANSETT BAY
FREQUENCY OF TIDAL FLOODING
FROM
HURRICANES AND STORMS
AT
PROVIDENCE, R. I.
CORPS OF ENGINEERS U.S. ARMY
NEW ENGLAND DIVISION
BOSTON, MASS. DEC. 1956
PLATE 16



	RANGE "A" 1□" = $ 40,000			RANGE "B" 1□" = $ 40,000			RANGE "C" 1□" = $ 20,000			TOTAL	
	AREA □"	ANNUAL LOSS	ANNUAL BENEFIT	AREA □"	ANNUAL LOSS	ANNUAL BENEFIT	AREA □"	ANNUAL LOSS	ANNUAL BENEFIT	ANNUAL LOSS	ANNUAL BENEFIT
NATURAL	13.45	$538,000		9.06	$362,400		2.44	$48,800		$949,200	
MODIFIED	0.13+	$5,400	$532,600			$362,400			$48,800		$943,800

nefit-cost Ratios

The economic justification for protective works has been determined by mparison of average annual benefits and costs. The New Bedford project, th a first cost of $17,200,000 has estimated annual charges of $691,000 com-red with annual benefits of $987,900 and a benefit-cost ratio of 1.4 to 1.0. e Fox Point project, estimated at $16,500,000 would have annual charges of 32,000, annual benefits of $1,733,000 and a benefit-cost ratio of 2.37 to 1.0.

SUMMARY AND CONCLUSIONS

Scientific studies of the meteorology and dynamics of hurricane surges, llection of data and basic research in tidal hydraulics form the groundwork r engineering application to the relatively new problems of preventing loss life and property from disastrous hurricane floods in coastal areas. Tech-cal studies have advanced to the point where it is possible to make practical plication towards preventing or mitigating flood disasters by such measures forecasts and warnings, evacuation, special construction within flood areas relocation outside of flood areas, protection by dikes, flood walls and tidal od barriers or breakwaters to protect against storm waves.

In addition to improved forecasting of hurricanes, which is already in ef-ct, recommendations have been made for construction of economically asible projects for hurricane flood protection of the Providence and the rraganset Bay area in Rhode Island and Massachusetts, and the New dford-Fairhaven area in Massachusetts. The high cost of hurricâne protec-e works in general, however, will limit their construction to areas of con-ntrated damages and locations where barriers will not seriously interfere th navigation and access to beaches and shore properties.

Problems of the height and frequency of hurricane surges and great storm ves are complex and vary widely along the beaches, promontories, tuaries and rivers of the New England Coast. The solution of the problem hurricane protection requires flood damage surveys and studies of past rricanes; determination of the maximum surge levels which may be ex-cted as a basis for sound design; calculation of higher levels in bays and tuaries due to wind set-up from hurricane winds blowing across shoal ters; determination of storm wave heights and the attenuation of waves th the effect of wave diffraction and refraction before impact on protective uctures. The solution of the many problems is accomplished by the appli-tion of tidal hydraulic calculations, use of hydraulic models to determine effect of dikes and barriers on the tidal regimen of bays and estuaries, d tests of hydraulic structures.

ACKNOWLEDGMENTS

The material presented herein is the result of studies conducted by the S. Army Engineer Division, New England, in cooperation with other Corps Engineers units, particularly the Beach Erosion Board, Washington, D. C. led by the Texas A & M Research Foundation and the Waterways Experiment tion at Vicksburg, Mississippi. A number of private engineering firms sisted in the studies, as well as Federal agencies such as the Weather reau, and the Coast and Geodetic Survey, Department of Commerce; the

Fish and Wildlife Service, Department of the Interior, and the Public Healt
Service of the Department of Health, Education and Welfare. State agencie
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e technical papers published in the past year are identified by number below. Technical-division orship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air sport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hy- ics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil anics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors , divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 ary 1956) papers were published in Journals of the various Technical Divisions. To locate papers in urnals, the symbols after the paper numbers are followed by a numeral designating the issue of a par- r Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which tes that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 T3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 39(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 M2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 56(ST3)[c], 1657(SM2)[c].

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: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 01(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 M3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

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scussion of several papers, grouped by divisions.